Seattle Public Utilities | Protecting Seattle's Waterways

Ship Canal Water Quality Project

Final Facility Plan Addendum

February 2018

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Facility Plan Acronyms

AACE	American Association of Cost Engineering
BMP	best management practice
cfm	cubic feet per minute
CIP	Capital Improvement Program
CN	Conservation Navigation
CO_2e	carbon dioxide equivalent
CSO Control Plan Amendment	*2012 King County Long-term Combined Sewer Overflow Control Plan Amendment*
CSO	combined sewer overflow
CWA	Clean Water Act
DAHP	Washington State Department of Archaeology and Historic Preservation
DDT	dichlorodiphenyltrichloroethane
DNRP	King County Department of Natural Resources and Parks, Wastewater Treatment Division
DOJ	U.S. Department of Justice
DSN	Discharge Serial Number
Ecology	Washington State Department of Ecology
EIS	environmental impact statement
EPA	U.S. Environmental Protection Agency
FAS	City of Seattle Department of Finance and Administrative Services
GDR	Geotechnical Data Report

GAR	Geotechnical Assessment Report
GMA	Growth Management Act
GSI	green stormwater infrastructure
HDPE	high-density polyethylene
I&I	inflow and infiltration
IG	Industrial General
JPA	Joint Project Agreement
LTCP	Long Term Control Plan, Volume 2 of the *Plan to Protect Seattle's Waterways*
Metro	Municipality of Metropolitan Seattle
MG	million gallons
MGD	million gallons per day
MH	maintenance hole
MTCA	Model Toxics Control Act
NIRR	no impact release rate
NPDES	National Pollutant Discharge Elimination System
NRHP	National Register of Historic Places
NW	northwest
O&M	operations and maintenance
OHWM	ordinary high water mark
PCMP	Post-Construction Monitoring Program
P&ID	process and instrumentation diagram
Plan	*Plan to Protect Seattle's Waterways*
QAPP	quality assurance project plan
RCW	Revised Code of Washington

RWSP	Regional Wastewater Services Plan
SCADA	supervisory control and data acquisition
SDOT	Seattle Department of Transportation
SEIS	Supplemental EIS
SEPA	State Environmental Policy Act
SERP	State Environmental Review Process
Ship Canal WQ Project	Ship Canal Water Quality Project
SMC	City of Seattle Municipal Code
SMP	Shoreline Management Plan
SPU	Seattle Public Utilities
SRF	State Water Pollution Control Revolving Loan Fund
SWMM	U.S. Environmental Protection Agency Storm Water Management Model
TBL	triple bottom line
TEPS	Tunnel Effluent Pump Station
UG	Urban General
UI	Urban Industrial
U.S.C.	United States Code
W	West
WAC	Washington Administrative Code
WISAARD	Washington Information System for Architectural and Archaeological Records Data
WQ	Water Quality

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1 Executive Summary

1.1 Background

The Ship Canal Water Quality Project (Ship Canal WQ Project) Facility Plan (Facility Plan) was finalized and submitted to the Washington Department of Ecology (Ecology) and the U.S. Environmental Protection Agency (EPA) in March 2017, meeting the requirements of Section S.8 of Seattle Public Utilities' (SPU's) National Pollutant Discharge Elimination System (NPDES) permit (WA0031682), Section V.B.14 of the City of Seattle's (City's) Consent Decree (Civil Action No. 2:13-cv-678; United States of America, 2013a), and Section V.B.15 of King County's Consent Decree (Civil Action No. 2:13-cv-677; United States of America, 2013b) as modified effective October 25, 2016. In their June 27, 2017, Final Facility Plan approval letter, Ecology and EPA noted that the Ship Canal WQ Project design team is refining the hydraulic model used to size the project and included a requirement to submit a Facility Plan Addendum with updated modeling results and any design changes resulting from the updated modeling. This revised Facility Plan serves as the required Facility Plan Addendum.

The Ship Canal WQ Project tunnel and associated facilities were originally sized and evaluated using EPA's Storm Water Management Model 5 (SWMM5), which was created to prepare SPU's Final Long-Term Control Plan (LTCP; Volume 2 of the *Final Plan to Protect Seattle's Waterways* [the Plan; SPU, 2015a]). Subsequently, MIKE URBAN (MU) models were developed to provide a consistent model platform for integrating the tunnel model with a newly developed King County Department of Natural Resources and Parks (DNRP) systemwide model. The new model provides performance assessment and evaluation of operational strategies that are consistent with the operations of the SPU and DNRP systems. Historical (1978 through 2009) and recent supplemental flow-monitoring and rainfall data through 2015 were used in the modeling effort.

The specific objectives of the modeling effort were to accomplish the following:

- Develop and calibrate MU models for SPU and DNRP combined sewer overflow (CSO) basins and facilities tributary to the Ship Canal WQ Project and the West Point Treatment Plant (West Point).
- Integrate the individual MU models into a single model that can be used to simulate the Ship Canal WQ Project and the North Interceptor to define operational strategies and evaluate the performance of the Ship Canal WQ Project.
- Provide a common platform between SPU and DNRP to assist with Ship Canal WQ Project start-up, commissioning, and operation.

- Calculate simulated performance statistics, including 20-year CSO discharge frequency averages, that will be used to demonstrate permit compliance during the early period of Ship Canal WQ Project operation.
- Provide a platform to assist with developing compliance reports throughout the life of the project.

Modeling results from this effort show an increase in minimum control volume from 15.24 million gallons (MG) to 17.5 MG, requiring an increase in the tunnel storage volume to operate and meet the control volume requirement. A decision was made to increase the size of the tunnel from 14-foot to 18-foot 10-inch nominal inner diameter to provide the increased storage volume, account for operational variabilities in the downstream Ballard Regulator and the North Interceptor, address the uncertainties of climate change, and help ensure long-term overflow frequency compliance. The larger diameter was also selected because it is constructed using a standard-size tunnel-boring machine used for transit tunnels, which is more cost-effective than using custom-sized boring machines. This Facility Plan Addendum describes the necessary improvements to implement this revised design approach and is based on an approximately 60-percent level of design.

This Facility Plan Addendum outlines sewer system improvements that are necessary to reduce CSOs from SPU's Ballard, Fremont, and Wallingford Basins and DNRP's 11th Avenue NW and 3rd Avenue W Basins. Figure 1-1 shows the Ship Canal WQ Project (also called the Ship Canal Project and formerly called the Shared West Ship Canal Tunnel Option) conceptual system illustration.

To help control CSOs from these areas, various storage and flow transfer concepts were evaluated in SPU's *Plan to Protect Seattle's Waterways* (the Plan; SPU, 2014a and 2015a) and DNRP's *2012 King County Long-term Combined Sewer Overflow Control Plan Amendment* (CSO Control Plan Amendment; King County, 2012a). The Ship Canal WQ Project was selected as the recommended option by both agencies. This Facility Plan describes the project components and other key considerations of the recommended option.

The City originally constructed a combined sewer system in the Ship Canal WQ Project area (project area), meaning that both sanitary sewage (sewage) and stormwater runoff are conveyed in the same pipes. The City, and later SPU, modified the sewer system over time. Some portions of the project area now have fully separated sewers, meaning that sewage and stormwater are collected and conveyed in separate systems. Other portions of the project area have partially separated sewers, meaning that stormwater from roof drains and foundations enters the sanitary sewer system, while stormwater from roadways enters a separate drainage system.

Much of DNRP’s system of regional interceptors was constructed before it was transferred to the Municipality of Metropolitan Seattle (Metro). Metro was formed through a referendum in 1958 and was the precursor regional wastewater agency to DNRP. Metro expanded the system in the 1960s and 1970s as part of a regional wastewater management strategy to reduce pollution to local water bodies. While some parts of DNRP's collection system are fully separated, the interceptors in the project area are considered combined sewers. Flows from the project area are conveyed to DNRP’s West Point Treatment Plant (West Point) for secondary treatment and ultimately discharged to Puget Sound. DNRP designed, sized, and built West Point as part of its CSO control planning to provide full secondary treatment for 300 million gallons per day (MGD) and to provide primary treatment and disinfection for an additional 140 MGD.

For combined and partially separated sewer systems, under wet-weather conditions, flows are a combination of sewage and stormwater. As long as the flows are within the capacity of the sewer system, the pipes convey all flows to West Point. However, if flows exceed the capacity of the sewer system, then the excess volume of sewage and stormwater discharges into receiving water bodies through CSO outfalls. For this project, these receiving water bodies are Lake Union, Lake Washington Ship Canal (Ship Canal), and Salmon Bay Waterway.

1.2 Regulatory Requirements

The following laws and regulations require that the City and King County limit CSOs to a 20-year moving average of no more than one untreated discharge per year per permitted outfall:

- **Revised Code of Washington (RCW) 90.48.480—**This law requires “the greatest reasonable reduction of combined sewer overflows.”
- **Washington Administrative Code (WAC) 173-245-020 (22)—**"’The greatest reasonable reduction’ means control of each CSO in such a way that an average of one untreated discharge may occur per year.”
- **City’s and King County’s NPDES permits and Consent Decrees—**These direct that a moving 20-year period be used for long-term averaging of the overflow frequency (United States of America, 2013a and 2013b).

SPU’s Ballard, Fremont, and Wallingford CSO outfalls (Outfalls 147, 150, 151, 152, and 174) and DNRP’s 3rd Avenue W (008) and 11th Avenue NW (004) outfalls exceed a 20-year moving average of one untreated discharge per year. These CSO outfalls are the focus of the CSO control measures described in this Facility Plan.

The following key terms relate to the volume and frequency requirements:

- **Control volume—**The amount of excess combined sewage that must be captured or intercepted upstream of the outfall such that a 20-year moving average of no more than one untreated discharge per year per outfall is achieved.
- **Storage volume—**The actual size of the facility that needs to be constructed to operate and meet the control volume requirement for all CSO basins being controlled under various conditions.

The storage volume is not necessarily the same as the control volume. Storage volume differs in that it depends on additional factors, including the following: 1) system hydraulics, 2) storage location, 3) control system, and 4) timing of the release of stored volumes to avoid impacts to downstream facilities.

The minimum control volume for the various project area basins, based on hydraulic and hydrologic modeling completed as part of the *Ship Canal Water Quality Integrated Hydraulic Model Report* (SPU, 2018), is 18.3 MG. The storage tunnel will have a nominal inner diameter of 18 feet, 10-inches.

DNRP and SPU of Seattle have entered into a Joint Project Agreement (JPA) that defines the joint project and the roles and responsibilities for each agency. DNRP's participation as a partner with SPU on the Ship Canal WQ Project has been approved and documented by modification to its Consent Decree with the EPA and Ecology, filed October 25, 2016 with the United States District Court, Western District of Washington (United States of America, 2016). Table 1-1 shows Consent Decree milestones dates for the joint project.

Table 1-1. Consent Decree Milestone Dates Relevant to Facility Plan

Ship Canal WQ Project Milestone	Ship Canal WQ Project Consent Decree Milestone Dates[a]
Submit Draft Engineering Report for Ship Canal WQ Project	March 31, 2017
Submit Final Engineering Report for Ship Canal WQ Project	December 31, 2017
Submit Draft (90 percent) plans and specifications to Ecology for Ship Canal WQ Project	March 31, 2020
Submit Final (100 percent) plans and specifications to Ecology for Ship Canal WQ Project	December 31, 2020
Start construction for Ship Canal WQ Project	July 1, 2021
Complete construction of Ship Canal WQ Project	December 31, 2025
Achieve control status for combined sewer basins controlled by Ship Canal WQ Project	December 31, 2026

[a] *Dates per the approved SPU's Final Plan (SPU, 2015a).*

1.3 Combined Sewer Overflow Control Options Development and Evaluation

The Draft SPU Long Term Control Plan (LTCP; Volume 2 of the *Plan to Protect Seattle's Waterways*; SPU, 2014a) detailed and evaluated the following four options for controlling CSOs in the Ballard, Fremont, and Wallingford neighborhoods as part of the Ship Canal WQ Project:

- SPU independent tanks and flow transfer projects (multiple storage tanks and flow transfers) and DNRP independent storage and flow transfer projects
- SPU independent tunnel and DNRP independent storage and flow transfer projects
- Combination of independent SPU and DNRP storage and flow transfer projects plus shared SPU and DNRP storage facilities
- Two shared SPU and DNRP tunnel projects

The recommended option for the Final SPU LTCP was identified using a triple bottom line (TBL) analysis of the highest-ranking options. TBL is an economic analysis technique that evaluates financial, social, and environmental costs, benefits, and risks of each option.

The shared SPU and DNRP Ship Canal WQ Project was found to be comparable in cost with other options to control CSOs, given the early stage of option development and uncertainty of cost estimating. The independent tanks and flow transfer projects option had similar capital costs based on this same level of cost uncertainty, but greater construction impacts and less future flexibility. SPU and DNRP agreed that the shared SPU and DNRP Ship Canal WQ Project was the preferred option for the Ship Canal area. This recommendation was included in the SPU Final Plan, which was approved by EPA and Department of Ecology on August 26, 2015. The following factors support this recommendation:

- The project will result in lower overall community impacts:
  - Significantly less truck traffic by using alternative rail or barge transportation of spoils and materials from the tunnel construction site,
  - Less surface excavation with the tunnel compared with tanks
  - Less conveyance with the tunnel, so less excavation occurring at surface excavation sites in the right-of-way compared with tanks
  - Shorter length of open cut pipeline construction disrupting street rights-of-way
  - Lower risk of encountering, handling, and remediating contaminated soils at the surface

- Both SPU and DNRP will gain greater operational flexibility and lower risk of compliance failure, provided by the aggregated storage volume serving the multiple CSOs in the project area. Centralized storage will offer benefit of reducing maintenance of DNRP and SPU infrastructure. Centralized storage also will offer the benefit of adding future capacity with fewer impacts.
- Less property will be required, and there will be less surface impact on required property; there will be an opportunity to surplus a significant portion of acquired property post-construction or to repurpose the property for beneficial public use.
- Most key property acquisition for the tunnel is already in progress by SPU, whereas independent tank-based storage would require a siting and property acquisition process for the DNRP tank and appurtenances. SPU would also need additional siting and property acquisition for independent tank-based storage. The anticipated duration of additional property siting and acquisition is a considerable risk to the overall compliance schedules for SPU and DNRP and is mitigated through the joint tunnel project.
- There will be greater opportunity for spoils disposal using barges or rail transport.
- Fewer pump stations will be required.

In addition, when viewed with greater attention toward nonmonetary considerations, the shared Ship Canal WQ Project tunnel option offers advantages over the independent tank-based storage and flow transfer options (see Table 9-2 in Chapter 9). Nonmonetary factors, such as social and environmental objectives, risk, and benefits were used to evaluate options in conjunction with other factors (see Tables 9-3 and 9-4 in Chapter 9).

The Facility Plan continues refining the recommended option from SPU's Final LTCP (SPU, 2015a). Additional engineering and scientific analyses were completed to better define physical project characteristics, assess environmental and community impacts, and refine project cost estimates.

1.4 Recommended Option

The Ship Canal WQ Project will provide offline storage of combined wastewater in a deep storage tunnel constructed between the Ballard and Wallingford CSO areas, on the north side of the Ship Canal. The project will control the Ballard CSO basins (Outfalls 150,151, and 152), Fremont (Outfall 174) and Wallingford (Outfall 147) CSO basins, DNRP 3rd Avenue W Overflow Structure (DSN008), and 11th Avenue NW Overflow Structure (DSN004). Figures 1-2 and 1-3 provides a plan view of the Ship Canal WQ Project location and components.

Flow monitoring data and hydraulic modeling analysis both indicate the Ship Canal CSO outfalls currently exceed the one untreated discharge per year regulatory standard.

Table 1-2 shows the predicted annual CSO frequency and volume and the control volume for each outfall that will be controlled by this project. These CSO statistics were derived from a series of 38-year simulations with calibrated hydraulic models and represent how the existing system performs under a wide variety of historical climate conditions.

Table 1-2. Long-Term Modeling Results (1978 to 2015): CSO Frequencies, Overflow Volumes, and Control Volumes [a]

Outfall	Average Number of CSO Events Per Year	Average Annual CSO Volume (MG)	Control Volume (MG)
Outfall 147	32.9	12.5	2.36
Outfall 174	12.5	6.7	1.56
3rd Avenue W [b]	9.5	16.1	4.07
11th Avenue NW [b]	12.5	13.3	3.14
Outfall 150/151	12.3	2.5	0.58
Outfall 152	53.3	40.1	6.55
TOTAL	**133.0**	**91.2**	**18.3**

[a] *The SPU control volumes account for future climate change and were identified through hydraulic modeling presented in the* Ship Canal Water Quality Project Integrated Modeling Report *(SPU, 2018). The DNRP control volumes are based on long term averages (from 1978 to 2015) as presented in the* Ship Canal Water Quality Project Integrated Modeling Report *(SPU, 2018). All volumes assume a future condition with increased I&I coming from separated system upstream of Matthews Park Pump Station.*

[b] *These are DNRP outfalls. 3rd Avenue W (referred to as DSN 008) and 11th Avenue NW (referred to as DSN 004).*

DSN discharge serial number

I&I infiltration and inflow

Table 1-3 shows the estimated frequency of CSO discharges after the Ship Canal WQ Project is implemented based on a 1978-to-2015 simulation conducted with integrated Tunnel, North Interceptor and SPU and King County Basin models (see *Ship Canal Water Quality Project Integrated Modeling Report* [SPU, 2018a]).

The main components of the Ship Canal WQ Project include the storage tunnel and appurtenances, conveyance facilities to convey SPU and DNRP CSO flows into the tunnel, and a pump station and effluent discharge pipeline to drain flows from the tunnel. The modeling results indicate the Ship Canal WQ Project would reduce CSO frequencies to less than or equal to one per year at each outfall.

These main components listed below were identified during design and/or conceptual planning and are shown on Figures 1-4 through 1-8:

- Storage tunnel will have an 18-foot 10-inch nominal inner diameter, measuring approximately 14,000 feet long.

Table 1-3. Current Predicted CSO Frequency with 18-Foot 10-Inch Nominal Inner-Diameter Tunnel Based on 1978 to 2015 Rainfall

Outfall	Average Number of CSO Events Per Year
147	0.4
174	0.4
3rd Avenue W	0.3
11th Avenue NW	0.3
150/151	0.4
152	0.4

Note: These CSO frequencies were identified through hydraulic modeling of the CSO control measure concepts presented in the Ship Canal Water Quality Project Integrated Modeling Report *(SPU, 2018). CSO frequencies are expected to increase over time due to climate change. SPU and DNRP will be conducting annual performance modeling during the post-construction monitoring period to assess and update the projected frequency. Overflow frequencies for 3rd Avenue W and 11th Avenue NW may increase, depending on tunnel operational strategies, but they will be less than one over a 20-year moving average per the regulatory standard and Consent Decree. SPU and DNRP are committed to ensuring the average CSO frequency at each outfall is no more than one over a 20-year moving average.*

  - The stored combined sewage in the storage tunnel will flow from the East Portal in Wallingford westward to the Tunnel Effluent Pump Station (TEPS) in Ballard.
  - The tunnel alignment is planned to be primarily in the street right-of-way along the north side of the Ship Canal.

- Seven diversion structures will divert combined sewage away from existing CSO outfalls to the tunnel.
- Five drop structures will convey combined sewage from the surface into the storage tunnel; four structures will have odor control systems.
- A pump station will be located at the West Portal with an average capacity of 32 MGD.

Conveyance facilities will include the following elements listed below; all conveyance sizing and quantities are approximate estimates based on current design to date, and actual diameters, lengths, and alignments of conveyance facilities will be finalized during the final design phase:

- Gravity sewer line to convey flows from SPU's diversion structure at Ballard Outfalls 151 (approximately 300 linear feet of up to 48-inch-diameter pipe) and 152 (approximately 2,200 linear feet of up to 60-inch-diameter pipe) to the tunnel drop shaft
- Gravity sewer line to convey flows from DNRP's diversion structure at 11th Avenue NW to the tunnel drop shaft (approximately 120 linear feet of 60-inch to 72-inch-diameter pipe)

- Gravity sewer line to convey flows from SPU's diversion structure at Fremont Outfall 174 to the tunnel drop shaft (approximately 200 linear feet of 30- to 36-inch-diameter pipe)
- Gravity sewer line to convey flows from DNRP's diversion structure at 3rd Avenue W (under the Ship Canal) to the tunnel drop shaft (approximately 740 linear feet 18- to 60-inch-diameter diameter pipe)
- Gravity sewer line to convey flows from SPU's diversion structure at Wallingford Outfall 147 to the tunnel drop shaft (approximately 1,000 linear feet of 36-inch-diameter pipe)
- Effluent discharge piping to convey flows from the TEPS to SPU's local sewer (approximately 100 linear feet of 24-inch-diameter pipe) and DNRP's existing Ballard Siphon wet-weather barrel forebay (approximately 1,900 linear feet of 48-inch-diameter effluent discharge pipe).

Gravity sewer lines to convey flows from SPU's diversion structures at Ballard Outfalls 151 and 152 and Wallingford Outfall 147 to the tunnel drop shafts have been excluded from the cost sharing agreement between SPU and DNRP and are the sole responsibility of SPU.

Following are key system components of the recommended option:

- Storage Tunnel—The storage tunnel will have an18-foot 10-inch nominal inner diameter. The tunnel will have a depth of 50 to 80 feet for most of the alignment, depending on the alignment revisions during the project final design. Flows will enter the storage tunnel by gravity and be pumped to the local SPU sewer and DNRP regional interceptor when downstream capacity in these systems is available. A flushing system at the East Portal will be used to clean the storage tunnel following operation to remove accumulated solids and debris.
- TEPS—Pump station with an average capacity of 32-MGD will be constructed at the West Portal, located within the deep shaft used to construct the tunnel. An above-grade building will provide secured access to the pump station dry-well and wet-well areas. An on-site diesel-powered generator will provide standby power. The TEPS will be designed for automated operation (unstaffed) and include safety and ventilation systems; electrical and control systems; access considerations and spatial considerations for on-site maintenance; permanent lifting equipment; and other operational systems required for safe long-term operations and maintenance (O&M) activities.
- Drop Shafts, Portals, and Vortex Drop Structures—Drop shafts and portals are finished facilities that will be located along the tunnel alignment providing conveyance functions and tunnel access. Located within the West Portal (wet well),

11th Avenue NW Drop Shaft, North 3rd Avenue/174 Drop Shaft, South 3rd Avenue Drop Shaft, and East Portal, vortex drop pipes will convey flows vertically downward from near-surface conveyance pipelines to the storage tunnel and allow movement of air to the odor control facilities. The drop shafts and portals will also provide access to the tunnel along the alignment for entry into the tunnel by maintenance staff as needed. Small standby generators located at the portals and most drop shafts will provide sufficient power for instrumentation and nearby control gates located at conveyance system diversion structures.

- Conveyance—This project will include structures needed to intercept combined sewer flows during storm events from the SPU and DNRP CSO basins. Gravity pipelines will convey flows to the storage tunnel. Diversion structures with control gates will direct water either into the tunnel or to existing outfalls. Conveyance elements will also include the TEPS effluent discharge pipeline that will convey flows to the Ballard Regulator Station and may include a new grit removal structure in the Basin 152 collection system upstream of the CSO interception structures. The primary anticipated construction method for conveyance pipes will be open-cut construction. Some sections will be constructed using trenchless method (microtunneling) to avoid extended surface impacts; cross under critical utilities, railroads, and streets; and to construct the 3rd Avenue W CSO connection under the Ship Canal to the North 3rd Avenue/174 Drop Shaft. Real-time controls, including automated adjustable gates, and level and flow sensors will be included at diversion structures to determine flows diverted to storage tunnel and to the existing outfalls.
- Odor Control—An odor control system incorporating a fan and activated carbon-scrubbing media to treat foul air from the tunnel will be located at the TEPS. An underground electrical and mechanical vault containing an activated carbon odor control system, mechanical, electrical, and control systems will be located at the 11th Avenue and North 3rd Avenue/174 Drop Shafts and at the East Portal. Odor control will be provided to South 3rd Avenue from the North 3rd Avenue/174 odor control system. Odor control at other locations will be evaluated during final design.
- Modifications to Existing System—Existing structures may be modified based on the results of hydraulic modeling that will be performed during design.

All conveyance sizing and quantities, including the storage tunnel, are estimates based on designing or planning to date. Actual diameters and lengths of conveyance facilities, tunnel depth and diameter, and size and function of associated facilities, including pumping systems, odor control, and standby power, will be determined during the project final design phase.

In addition to the key system components described above, the project will incorporate the following elements:

- 24th Avenue NW Pedestrian Pier Improvements—A considerable portion of tunnel construction spoils and other waste materials will be transported to a disposal site using barges. The existing 24th Avenue NW Pedestrian Pier located adjacent to the West Portal will require reconstruction in its current location to accept the anticipated loading equipment required for the effective use of barges. When the project is completed, the reconstructed pier will be converted back to a public amenity.
- Outfall 151 Rehabilitation—The existing 18-inch-diameter wood-stave Outfall 151 is in poor condition, and rehabilitating it during Ship Canal WQ Project construction will be less disruptive to the community than constructing a separate rehabilitation project. SPU plans to replace both the existing Outfall 151 and the existing 30-inch-diameter Outfall 150 with a single 48-inch-diameter outfall. This replacement Outfall 151 will be installed under the new 24th Avenue NW Pedestrian Pier.

After the Ship Canal WQ Project is constructed and operating, CSOs will occur only during extreme storm events when the capacity of the tunnel is exceeded or when conveyance capacity to the tunnel is exceeded. Stored flows will drain from the tunnel to West Point for treatment after rainfall ends and/or conveyance capacity is available.

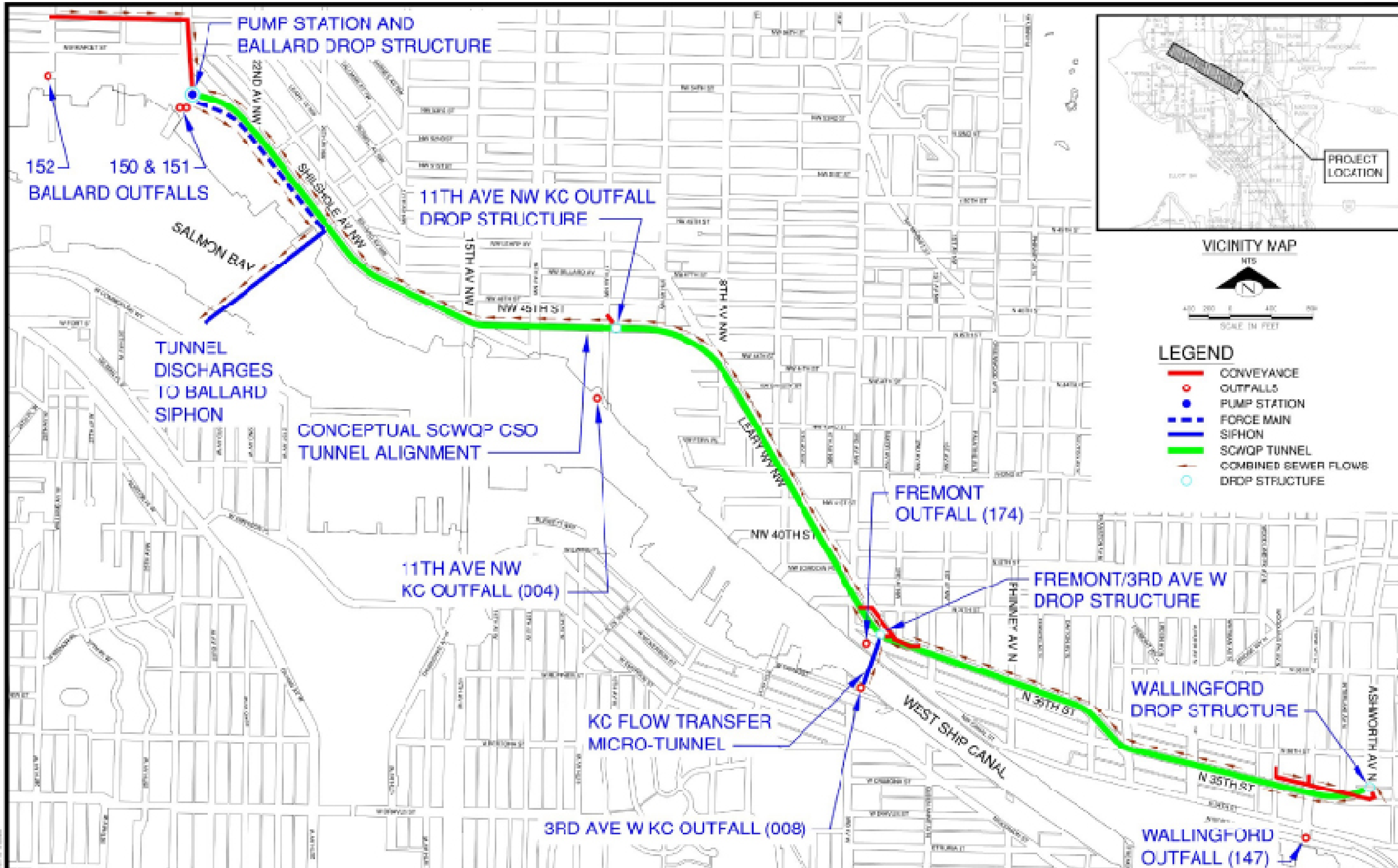
Table 1-4 shows the projected annual cash flow for the Ship Canal WQ Project based on the project schedule included in Appendix A. The schedule and cash flow were initially developed in December 2016 and are subject to change as the project schedule is updated. The dollars are escalated to the year in which the costs are projected to occur. For example, the amounts for 2017 are expressed in 2017 dollars while the amounts for 2018 are expressed in 2018 dollars. A 2-percent annual inflation rate was used for the cost escalation. The dollars are based on the total cost projection presented in Table 11-1 in Chapter 11 (14-foot-diameter tunnel basis). The cost share between SPU and DNRP is discussed in Chapter 12.

Table 1-4 Projected Annual Cash Flow for the Ship Canal Water Quality Project [a]	
Year	**Annual Cash Flow [b]**
Prior Years [c]	$38,200,000
2017	$10,000,000
2018	$15,900,000
2019	$24,800,000
2020	$71,000,000
2021	$101,600,000
2022	$72,500,000
2023	$47,300,000
2024	$32,400,000
2025	$9,700,000
TOTAL	**$423,400,000**

[a] *This cash flow is based on the estimated costs of the originally proposed 14-foot-diameter tunnel. An updated cost estimate and cash flow for the 18-foot 10-inch diameter tunnel is expected to be completed in 2018.*

[b] *The amounts in future years (i.e., 2017 and beyond) are adjusted for inflation*

[c] *The amount from prior years is based on actual dollars spent.*



Seattle Public Utilities — Ship Canal Water Quality Project Final Facility Plan	Conceptual Ship Canal Water Quality Project (SCWQP) System Illustration	
	JANUARY 2018	FIGURE 1-1

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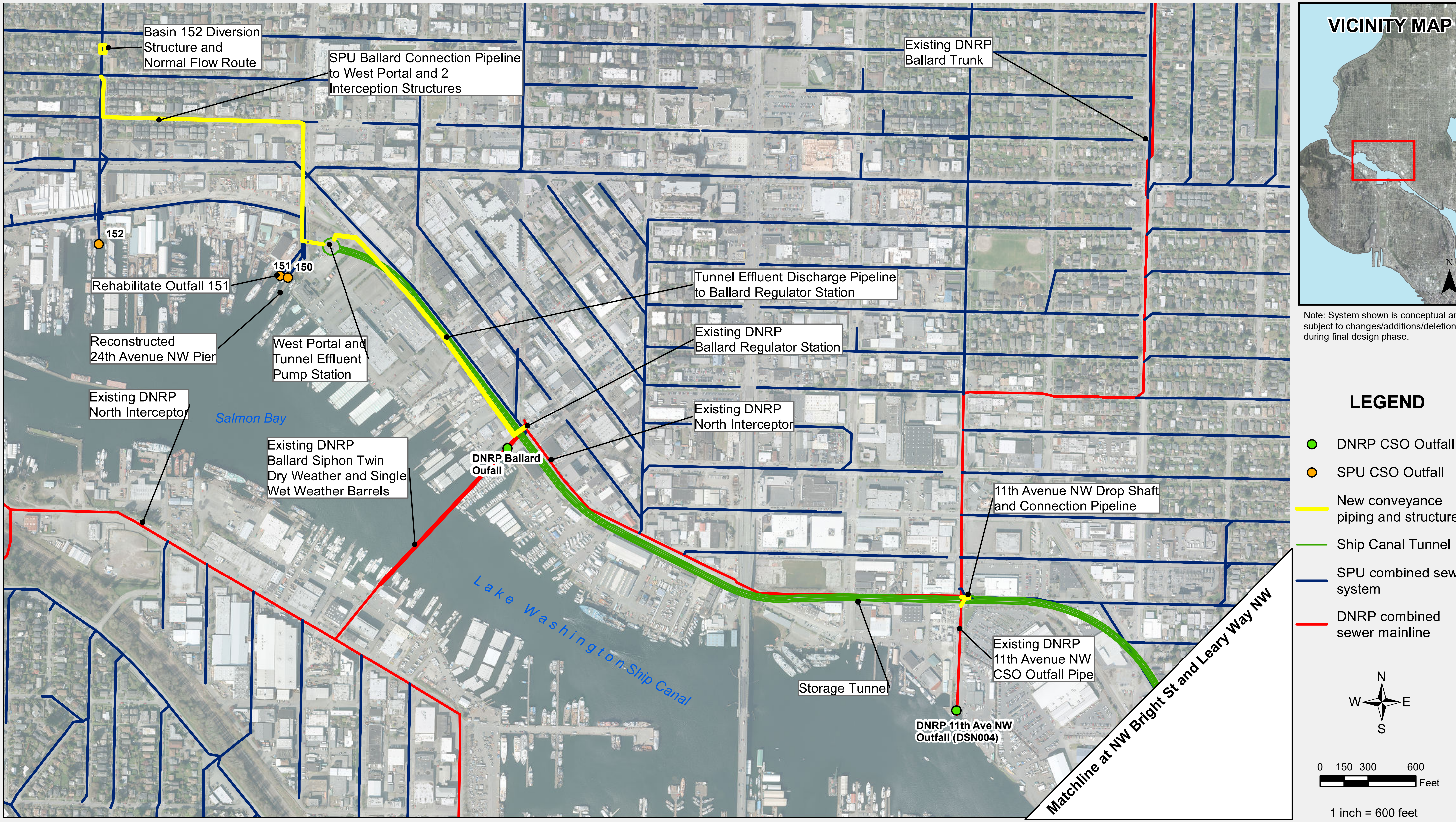

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Ship Canal Water Quality Project
Final Facility Plan

RECOMMENDED OPTION OVERVIEW WEST

JANUARY 2018 | **FIGURE 1-2**

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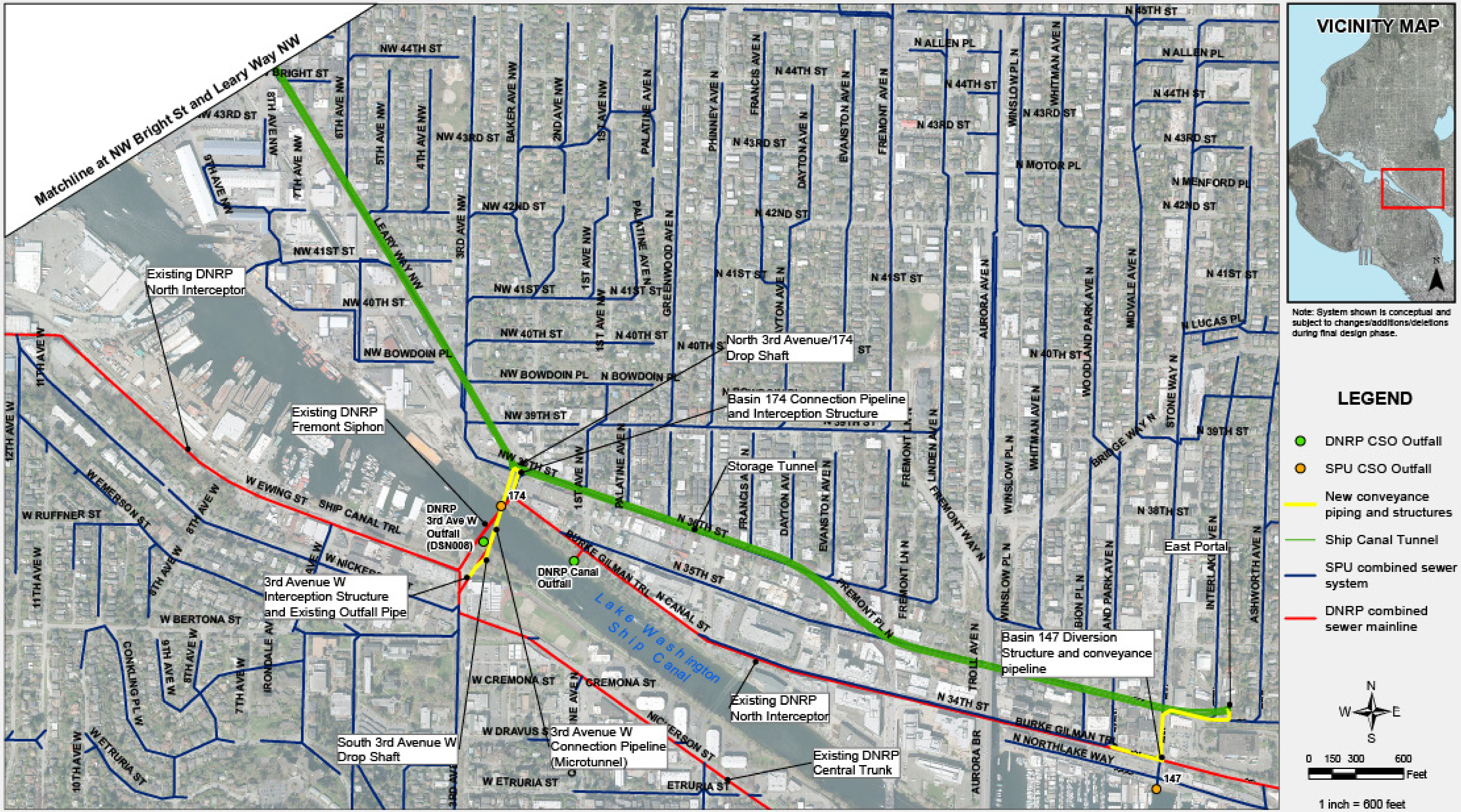

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RECOMMENDED OPTION OVERVIEW EAST

JANUARY 2018 | **FIGURE 1-3**

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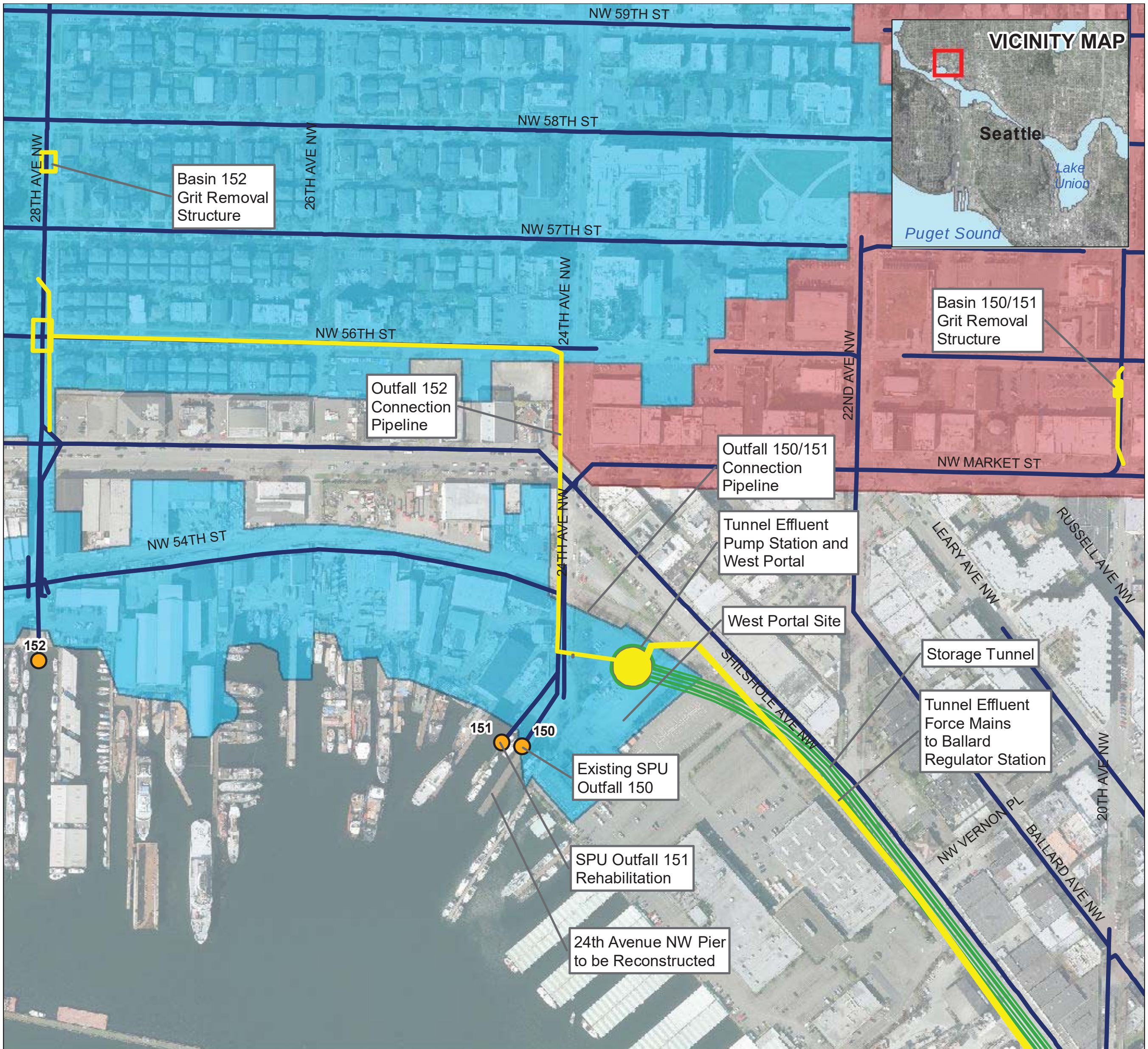

Note: System shown is conceptual and subject to changes/additions/deletions during final design phase.

0 200 400 Feet

N W E S

LEGEND

- New conveyance piping and structures
- SPU CSO Outfall
- Ship Canal Tunnel
- SPU combined sewer system
- SPU Basin 150/151 (Ballard)
- SPU Basin 152 (Ballard)

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West Portal and Ballard Conveyance Area

JANUARY 2018 | **Figure 1-4**

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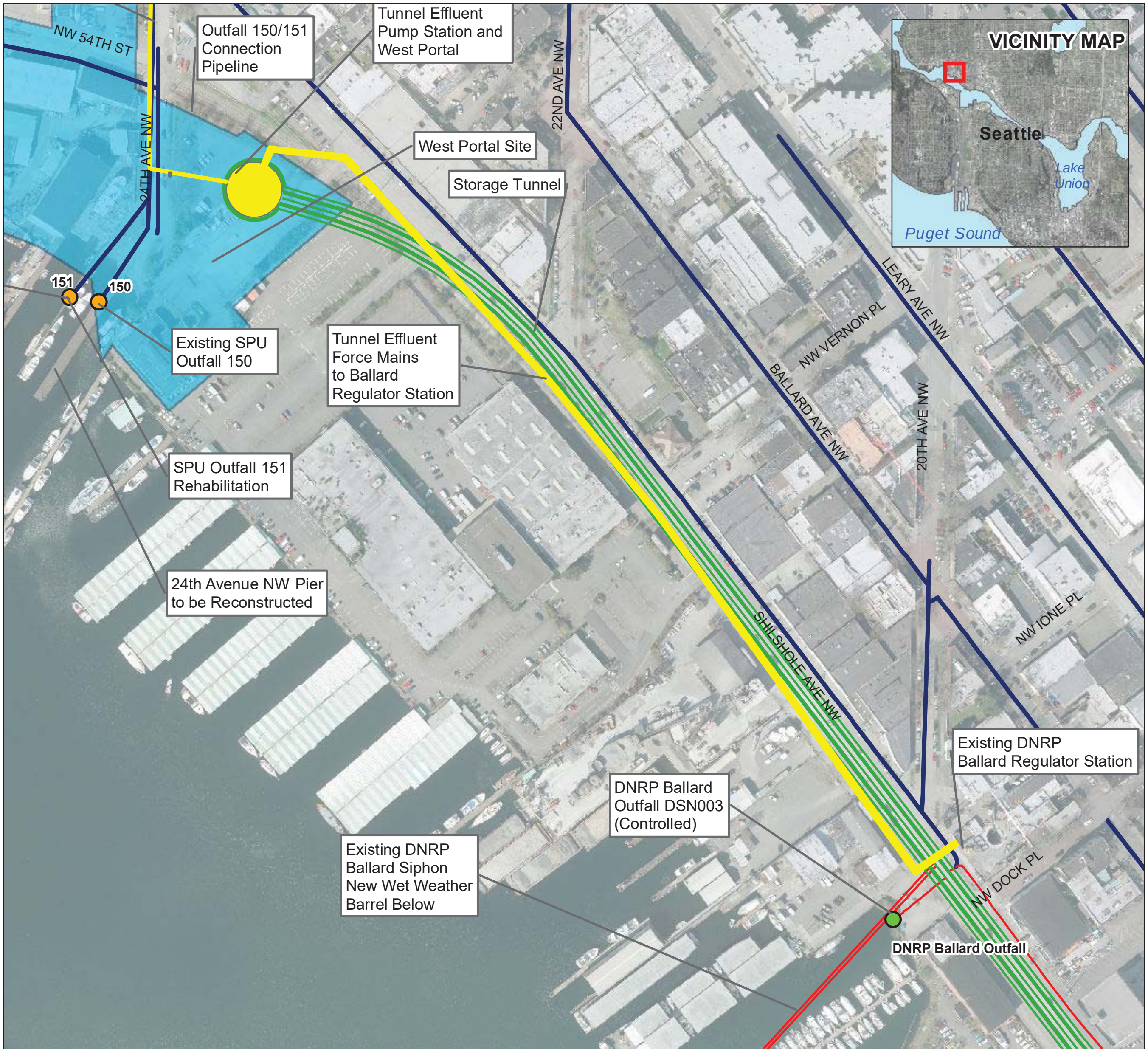

Note: System shown is conceptual and subject to changes/additions/deletions during final design phase.

0 150 300 Feet

N W E S

LEGEND

- DNRP CSO Outfall
- SPU CSO Outfall
- New conveyance piping and structures
- Ship Canal Tunnel
- DNRP combined sewer mainline
- SPU combined sewer system
- SPU Basin 152 (Ballard)

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Final Facility Plan

Effluent Discharge Pipeline Conveyance Overview

JANUARY 2018

Figure 1-5

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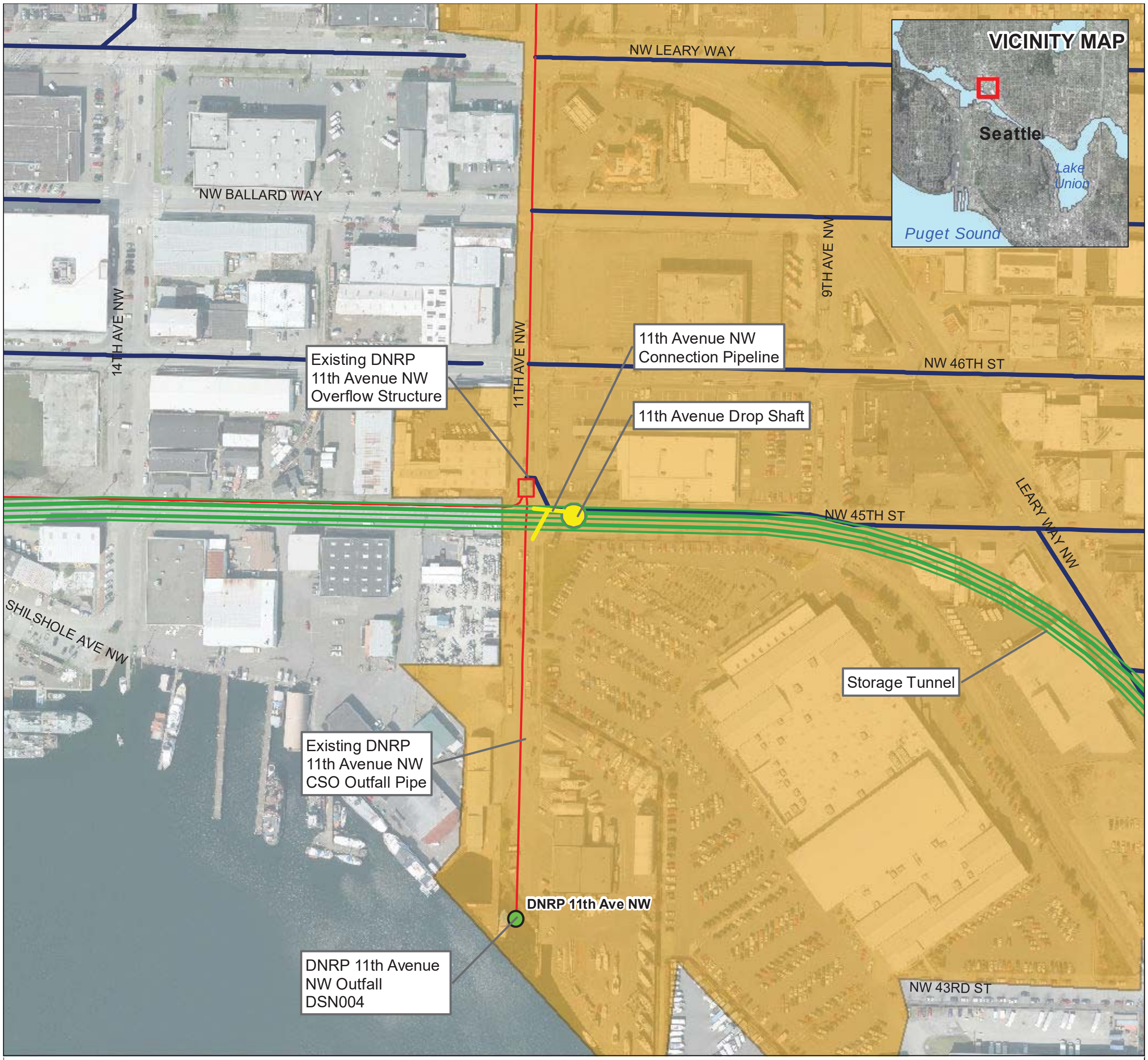

Note: System shown is conceptual and subject to changes/additions/deletions during final design phase.

0 150 300 Feet

N E S W

LEGEND

- DNRP CSO Outfall
- New conveyance piping and structures
- Ship Canal Tunnel
- DNRP combined sewer mainline
- SPU combined sewer system
- DNRP 11th Ave NW

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11th Avenue Drop Shaft and Conveyance Area

JANUARY 2018 **Figure 1-6**

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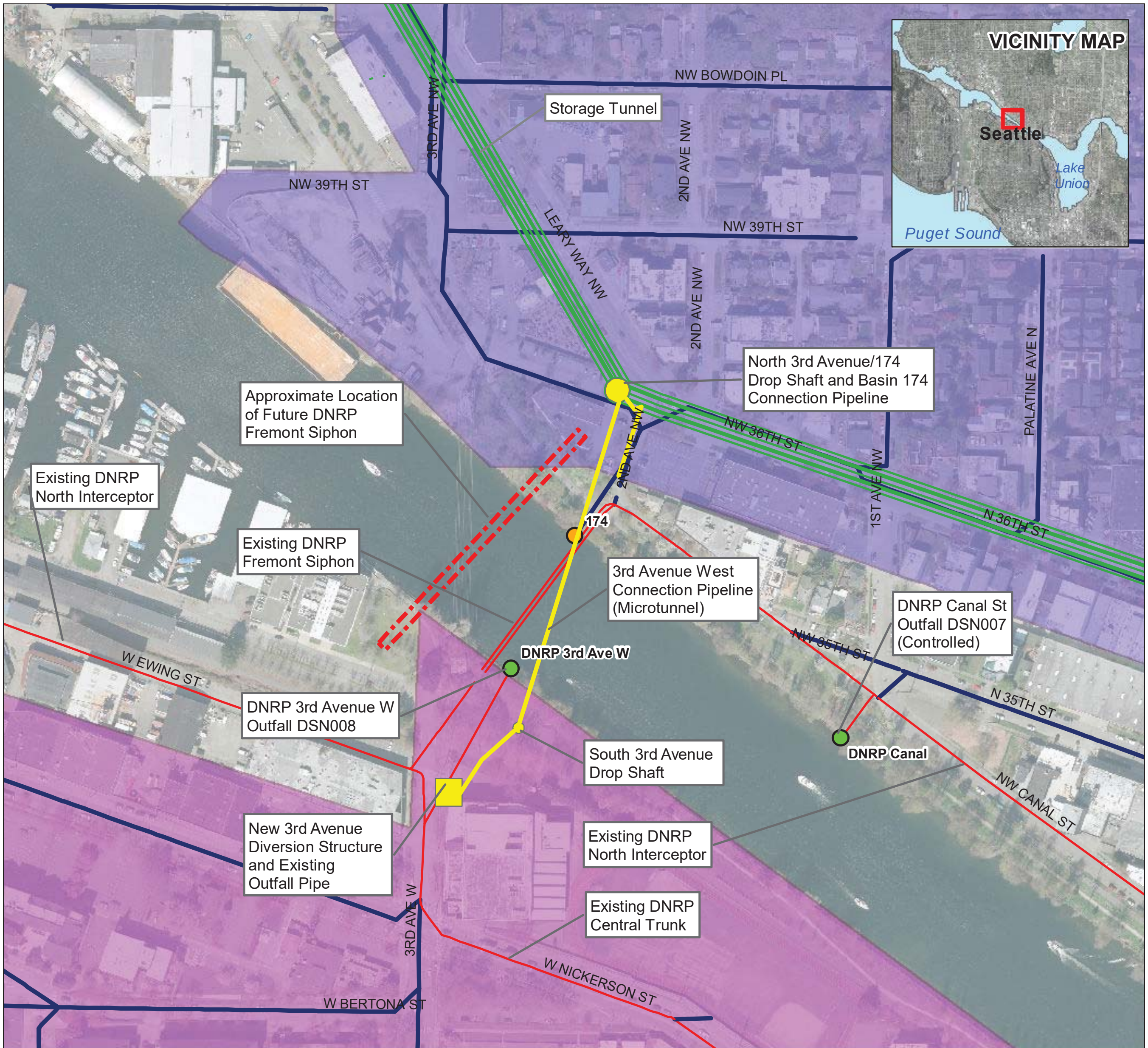

Note: System shown is conceptual and subject to changes/additions/deletions during final design phase.

LEGEND

- DNRP CSO Outfall
- SPU CSO Outfall
- New conveyance piping and structures
- Ship Canal Tunnel
- Future siphon
- DNRP combined sewer mainline
- SPU combined sewer system
- DNRP 3rd Ave W
- SPU Basin 174 (Fremont)

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North 3rd Avenue/174 Drop Shaft and Conveyance Area

JANUARY 2018 | **Figure 1-7**

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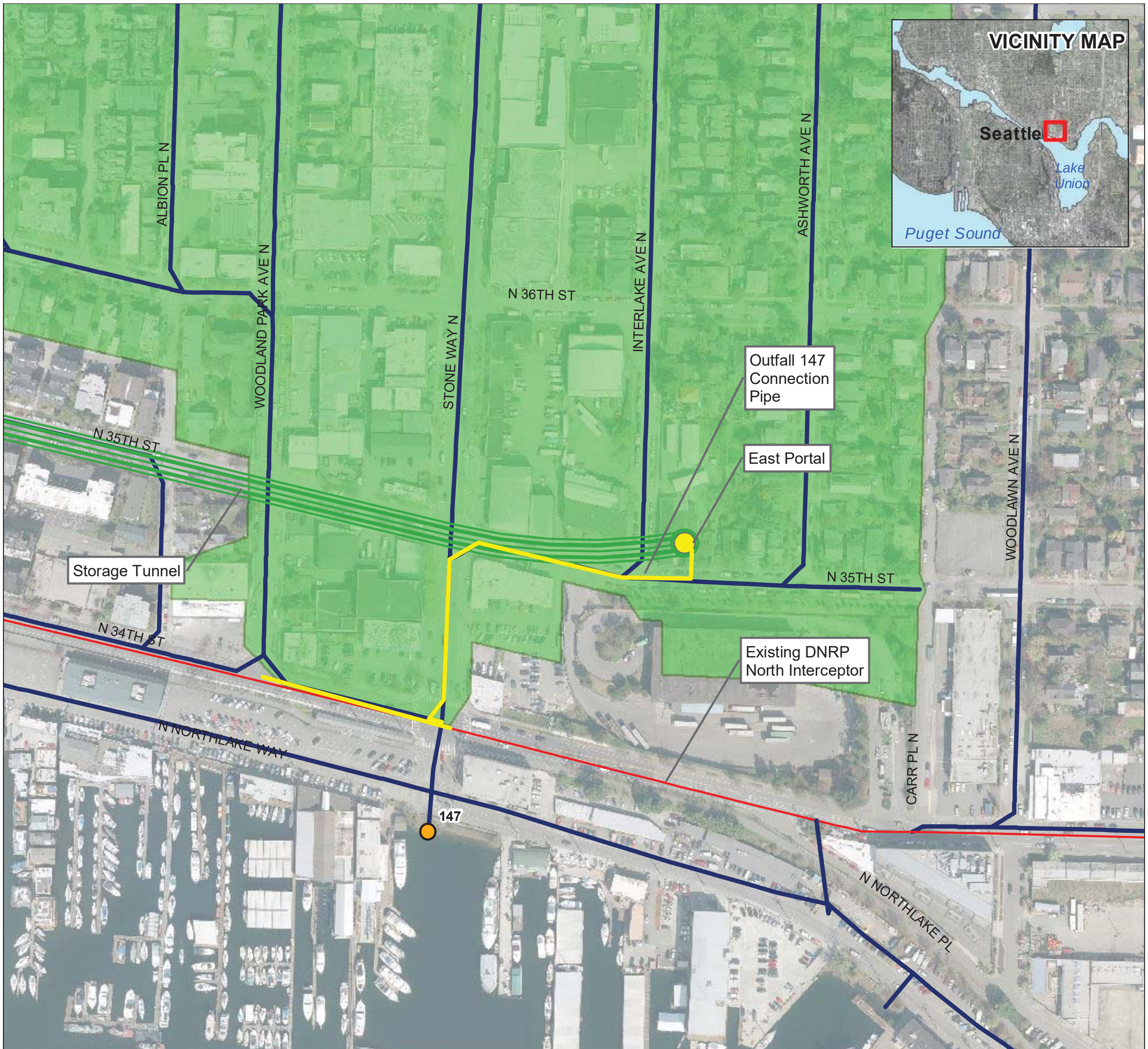

Note: System shown is conceptual and subject to changes/additions/deletions during final design phase.

0 150 300 Feet

N W E S

LEGEND

- SPU CSO Outfall
- New conveyance piping and structures
- Ship Canal Tunnel
- DNRP combined sewer mainline
- SPU combined sewer system
- SPU Basin 147 (Wallingford)

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East Portal and Basin 147 Conveyance Area

JANUARY 2018 | **Figure 1-8**

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6 Historical Combined Sewer System Flows

This chapter characterizes the historical combined sewer flows from the project area, which include large portions of North Seattle and CSO outfalls in the Wallingford, Fremont, Ballard, and north Queen Anne neighborhoods. SPU and DNRP have developed an understanding of sewer system flows through a combination of flow monitoring and hydraulic modeling. Together, the monitoring data and modeling results produce the information necessary to characterize system performance, understand hydraulic issues, and evaluate and design CSO control projects.

6.1 Monitored Basin Flows

Flow and level data were collected in the Ship Canal WQ Project area and used to characterize system hydraulics and calibrate hydraulic models. The monitoring program consists of permanent stations (including SCADA locations) that provide CSO discharge monitoring and assist in system operation and temporary monitoring sites that supplement system hydraulics characterization. Together, permanent station and temporary monitoring data were used to create a more robust model calibration to support calculating CSO control volumes. Table 6-1 lists the average dry-weather flow for each CSO area. Wet-weather flow conditions are described in the subsequent sections.

Table 6-1. Dry-Weather Flows in the Ship Canal Water Quality Project Basins

CSO Basin	Average Dry-Weather Flow (MGD)
147	0.4
174	0.6
3rd Avenue W	5.3
11th Avenue NW	5.2
150/151	0.4
152	0.9

6.1.1 Temporary Flow Monitoring

During development of the *Plan to Protect Seattle's Waterways* (SPU, 2015a), temporary flow monitors were installed in the Fremont and Wallingford neighborhoods (Basins 174 and 147) and Ballard neighborhood (Basins 150/151, and 152). The *Flow Monitoring Summary Report* (SPU, 2010b) describes the flow monitoring program for the Fremont/Wallingford and Ballard areas and the monitoring data collected from October

2008 through May 2010. The report documents the flow data results, quality assessment, and related information. The project team used this information to develop dry-weather flows; document wet-weather influences on system flows; characterize hydraulic performance of the system, including weirs and other hydraulic structures; and calibrate and validate the hydraulic models. DNRP used temporary flow monitoring and level data to support the Central Trunk model calibration and to estimate CSO control volumes at the 3rd Avenue W Outfall. Seven temporary meters installed in SPU and DNRP sewers were used to estimate dry-weather flows, calibrate the model's wet weather response, and estimate diversions from the Central Trunk system (tributary to the 3rd Avenue Overflow Structure) to the Mercer Tunnel system. Temporary level monitoring data helped verify DNRP's hydraulic model calibration at the 11th Avenue NW Overflow Structure weir. This verification was part of the Ballard Regulator Station Siphon Design Project completed by DNRP in 2013.

Table 6-2 describes how temporary flow monitors were used in the hydraulic models. Figures 6-1 through 6-4 (reproduced from the hydraulic model reports; SPU, 2012a and 2012b) show a schematic view of monitoring locations and dry-weather flows in each basin. Figure 6-5 provides a schematic view of the temporary monitoring program in the 3rd Avenue W area. No figure is included for 11th Avenue NW area because this model was calibrated to permanent monitoring data collected downstream at the Ballard Regulator Station.

6.1.2 No Impact Release Rate

The Ship Canal WQ Integrated Model simulates the flow and depth of flow in the DNRP North Interceptor. This model allows for abandoning the No-Impact-Release-Rate (NIRR) used in the LTCP to control drainage from the tunnel, because it now uses the simulated depth of flow in the DNRP North Interceptor. Previous modeling for the LTCP used NIRR, which constitutes a set of time series data obtained from models, identifying available capacity at a specific point in the DNRP system after DNRP's future CSO control projects are on-line. The NIRR estimates when and how SPU can drain a storage facility or transfer captured CSOs to a specific point in the DNRP system without adversely impacting DNRP facilities. Predicted performance of the Ship Canal WQ Project was analyzed using NIRRs in SPU's *Plan to Protect Seattle's Waterways*, Volume 2: LTCP, Appendix L (SPU, 2015a).

6.1.3 Permanent Flow Monitoring

SPU and DNRP operate and maintain permanent monitoring equipment to identify overflow frequency and estimate discharge volumes at each CSO outfall. SPU and DNRP report discharge duration, discharge volume, and weather-related information (precipitation and storm duration) on a monthly and annual basis, in accordance with their NPDES waste discharge permits.

Table 6-2. Temporary Monitoring Summary for Model Calibration

CSO Basin	Number of Temporary Flow Monitors	Flow Monitor Usage
147	8	Hydrology/hydraulic calibration = 6 Establish boundary condition = 2
174	6	Hydrology/hydraulic calibration = 5 Establish boundary condition = 1
3rd Avenue W	7	Hydrology/hydraulic calibration = 6 Characterize system operation = 1
11th Avenue NW [a]	6	Hydrology/hydraulic calibration = 6 Hydraulic verification at regulator = 1
150/151	4	Hydrology/hydraulic calibration = 3 Establish boundary condition = 1
152	16	Hydrology/hydraulic calibration = 13 Used to support GSI [b] analysis = 2 Establish boundary condition = 1

[a] Temporary monitoring was conducted at the 11th Avenue NW Overflow Structure weir to verify the hydraulic model performance as part of the Ballard Siphon design project.

[b] Data from flow-monitoring equipment installed in Ballard determined the fraction of wet-weather flow entering the system from different sources (for example, rooftops versus public right-of-way connections).

GSI green stormwater infrastructure.

The hydraulic models for each basin utilized the permanent monitoring data at the CSO structures to calibrate and/or verify the model predictions. For example, the permanent monitoring data at SPU's CSO structures were used to estimate hydraulic losses within these CSO structures and finalize the hydraulic calibration. DNRP's models used SCADA information at the 3rd Avenue Overflow Structure, 11th Avenue NW Overflow Structure, and Ballard Regulator Station (downstream of 11th Avenue NW) to support model calibration and verification and to supplement temporary monitoring data collected in the area. Figure 6-5 shows permanent flow monitoring locations for DNRP's 3rd Avenue W CSO Basin. DNRP does not have permanent flow monitoring data for the 11th Avenue NW CSO Basin.

Table 6-3 summarizes the reported CSO discharge records from 2010 through 2014 for the seven outfalls addressed by the Ship Canal WQ Project, as reported annually to Ecology. The table indicates that each outfall overflows several times per year and shows the relative CSO discharge frequency and volume among the outfalls.

Table 6-3. Ship Canal Water Quality Project Basins Reported CSO Frequency and Volumes 2010-2014

Outfall	Total Number of CSO Events	Average Number of CSO Events Per Year	Average Annual CSO Volume (MG)
147	226	45.2	12.9
174	67	13.4	7.5
3rd Avenue W (DSN008)	45	9.0	8.2
11th Avenue NW (DSN004)	92	18.4	11.6
150/151	133	26.6	3.1
152	265	53.0	37.5
TOTAL	**828**	**165.6**	**80.8**

6.1.4 Rain Gauges

SPU has operated a citywide network of rain gauges since the late 1970s. Figure 6-6 shows the locations of these and DNRP's gauges and outlines of the contributing areas for each CSO outfall. Data from SPU's rain gauges 1, 7, 8, 9, 11, and 20 were used to model SPU's and DNRP's sewers in the Ship Canal WQ Project basins.

6.2 Modeled Basin Flows and Control Volumes

The hydraulic models of the Ship Canal WQ Project basins were developed and progressively refined to support the understanding of the combined sewer system, wet-weather flows, and CSO events and then later evaluate alternative measures for CSO control. SPU's and DNRP's modeling efforts are documented in the following reports:

1. SPU's hydraulic model reports (SPU, 2012a and 2012b) describe the development of basin models, including flow monitoring data and special hydraulic structures. The reports also cover model calibration and validation. Volume 2 (2012a) describes the Ballard model for Outfalls 150, 151, and 152, and Volume 5 (2012b) describes the Fremont and Wallingford models for Outfalls 174 and 147.

2. The *Plan to Protect Seattle's Waterways*, Volume 2: LTCP, Section 2.6 and Appendix G (SPU, 2015a) describe the long-term model simulations, uncertainty analysis, and control volumes for SPU's CSO outfalls.

3. The *Plan to Protect Seattle's Waterways*, Volume 2: LTCP, Appendix L (SPU, 2015a) describes the analysis of specific CSO control options, such as tanks and tunnels. The document includes standalone control strategies for SPU outfalls and joint projects for SPU and DNRP outfalls. The CSO models include DNRP's no NIRRs, which are used to determine when and how storage facilities can be drained

during and after storm events based on available capacity in the DNRP system, without adversely impacting DNRP facilities.

4. King County's Long-Term CSO Control Plan Amendment, Appendix B (King County, 2012a), describes the hydraulic modeling and monitoring approach to computing control volumes and evaluating CSO control options for the 3rd Avenue W and 11th Avenue NW outfalls.

5. *Ship Canal Water Quality Project Integrated Modeling Report* (SPU, 2018; Appendix D), describes the hydrologic and hydraulic modeling of the Ship Canal WQ Project storage tunnel integrated with the DNRP North Interceptor and associated inflows from the University of Washington area to the West Point Treatment Plant (WPP) to confirm compliance with regulations and to guide final design of the joint project.

The following sections summarize the modeling results, which demonstrate that the recommended project will bring the Ship Canal WQ Project basin outfalls into compliance. Table 6-4 lists the predicted annual CSO frequency and volume and the control volume for each of the outfalls. These CSO statistics were derived from a series of 38-year simulations with calibrated hydraulic models and represent how the existing system performs under a wide variety of historical climate conditions. Each outfall experiences several CSO discharges per year.

Table 6-4. Long-Term Modeling Results (1978 to 2015): CSO Frequencies, Overflow Volumes, and Control Volumes

Outfall	Average Number of CSO Events Per Year	Average Annual CSO Volume (MG)	Control Volume (MG)
147	32.9	12.5	2.36
174	12.5	6.7	1.56
3rd Avenue W	9.5	16.1	4.07
11th Avenue NW	12.5	13.3	3.14
150/151	12.3	2.5	0.58
152	53.3	40.1	6.55
TOTAL	**133.0**	**91.2**	**18.3**

Note: The SPU control volumes account for future climate change and were identified through hydraulic modeling presented in the Ship Canal Water Quality Project Integrated Modeling Report *(SPU, 2018), with boundary conditions provided for the DNRP combined sewer conveyance system. The DNRP control volumes are based on long-term averages (from 1978 to 2015) as presented in the* Ship Canal Water Quality Project Integrated Modeling Report *(SPU, 2018).All volumes assume a future condition with increased I&I coming from the separated system upstream of Matthews Park Pump Station.*

The model will be updated during detailed design, and the project design will be updated as appropriate. The recommended project (see Chapter 10) is a shared SPU and DNRP deep tunnel that will store combined sewer flows from the Ship Canal WQ Project basins during storms and return these flows to DNRP's regional conveyance system when capacity is available. The tunnel diameter was selected to accomplish the following:

- Store the increased control volume.
- Provide storage volume necessary to account for operational variabilities in the downstream Ballard Regulator and the North Interceptor.
- Address the uncertainties of climate change.

Ensure long-term overflow frequency compliance. The larger diameter was also selected because it is constructed with a standard-size tunnel-boring machine commonly used for transit tunnels, which is more cost-effective than using custom-sized boring machines. Table 6-5 shows the estimated frequency of CSO discharges after the project is implemented based on a 20-year modeled simulation (1996 to 2015) conducted with calibrated CSO models using historical rainfall data and Ship Canal Water Quality Integrated Hydraulic Model (see the *Ship Canal Water Quality Project Integrated Modeling Report*, [SPU, 2018], for additional details). The CSO frequencies are expected to increase over time due to climate change. SPU and DNRP will conduct annual performance modeling to assess and update the projected frequency. SPU and DNRP are committed to ensuring that the average annual CSO frequency at each outfall is no more than one over a 20-year moving average.

Table 6-5. Current Predicted CSO Frequency with 18-Foot 10-Inch Nominal Inner-Diameter Tunnel: Based on 1996 to 2015 Rainfall

Outfall	Average Number of CSO Events Per Year
147	0.4
174	0.4
3rd Avenue W	0.3
11th Avenue NW	0.3
150/151	0.4
152	0.4

Note: These CSO frequencies were identified through hydraulic modeling of the CSO control measure concepts in the Ship Canal Water Quality Project Integrated Modeling Report *(SPU, 2018). The CSO frequencies likely will increase over time due to climate change. SPU and DNRP will be conducting annual performance modeling during the post construction monitoring period to assess and update projected frequency. Overflow frequencies for 3rd Avenue W and 11th Avenue NW may increase depending on tunnel operational strategies, but they will be less than one over a 20-year moving average per the regulatory standard and Consent Decree. SPU and DNRP are committed to ensuring the average annual CSO frequency at each outfall is no more than one over a 20-year moving average.*

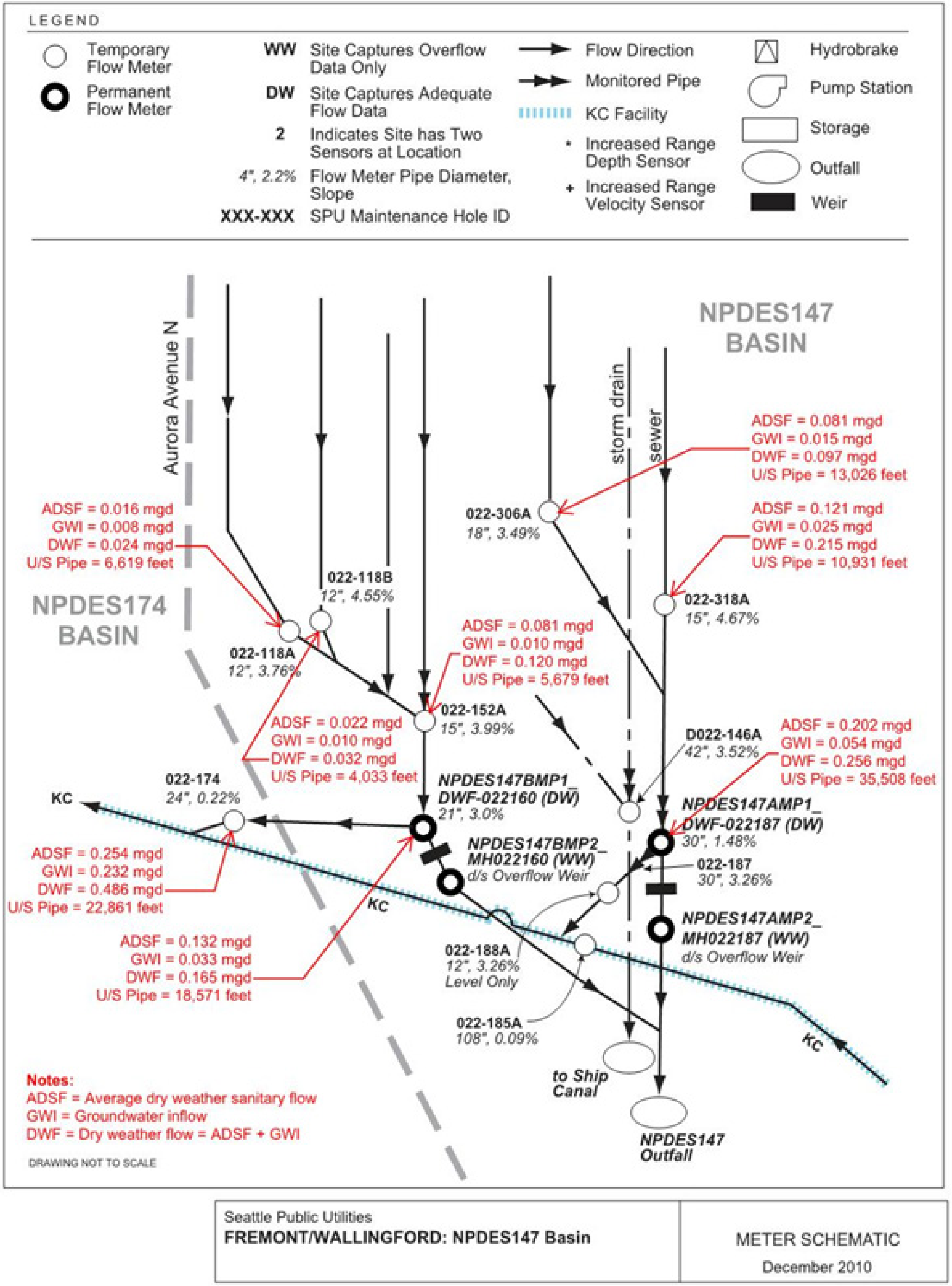


Figure 6-1. Schematic Dry-Weather Flow Summary for Basin 147

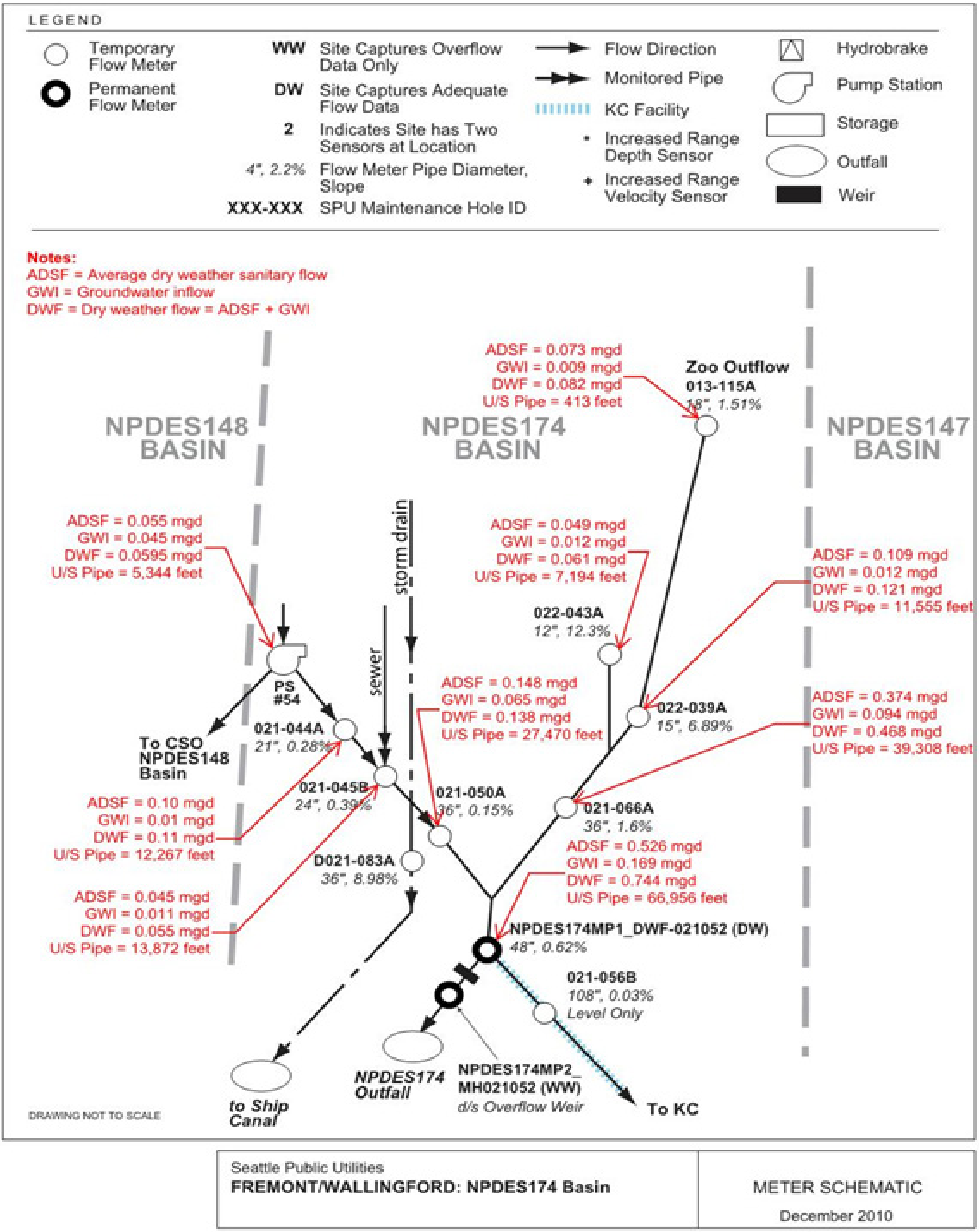


Figure 6-1. Schematic Dry-Weather Flow Summary for Basin 174

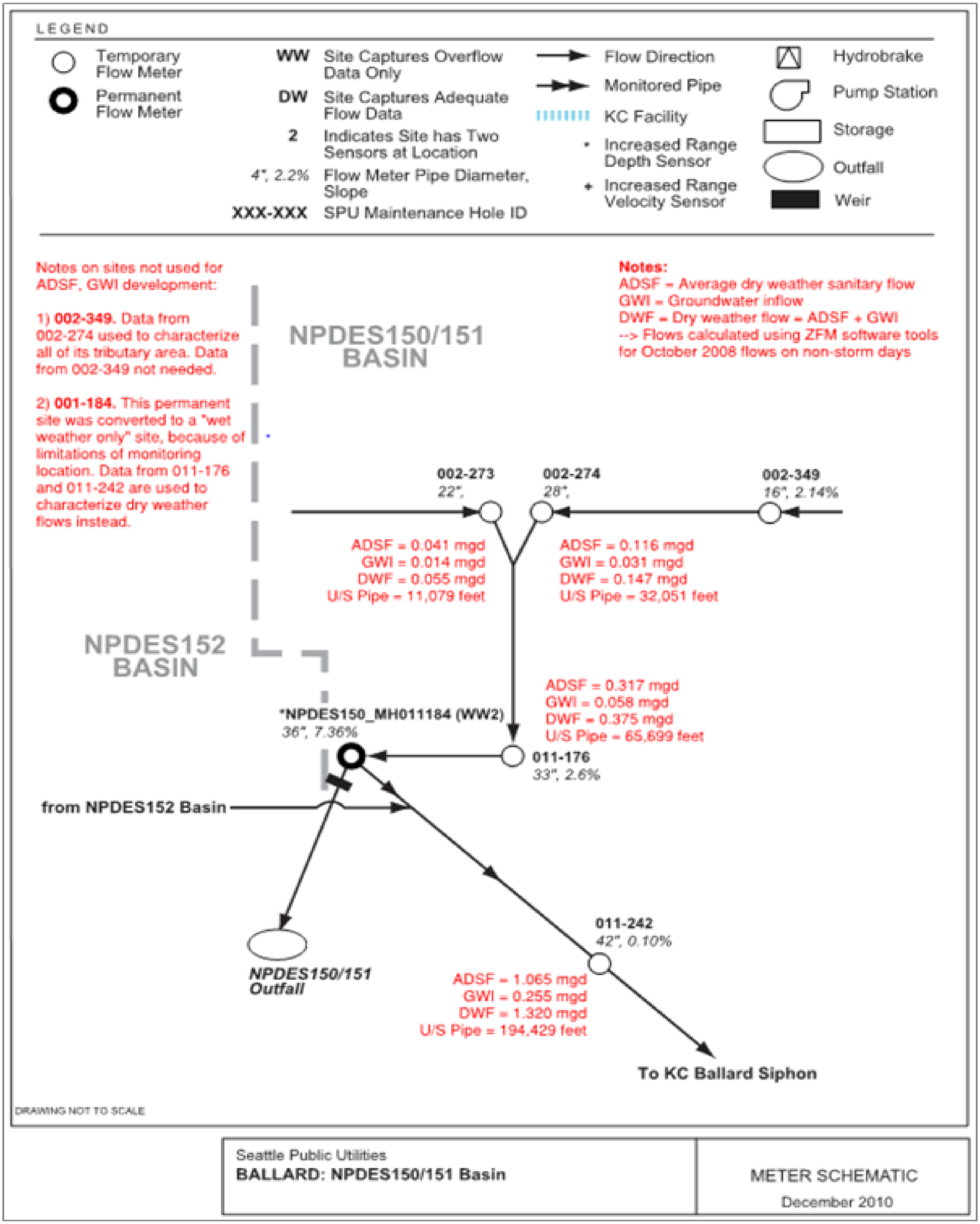


Figure 6-3. Schematic Dry-Weather Flow Summary for Basin 150/151

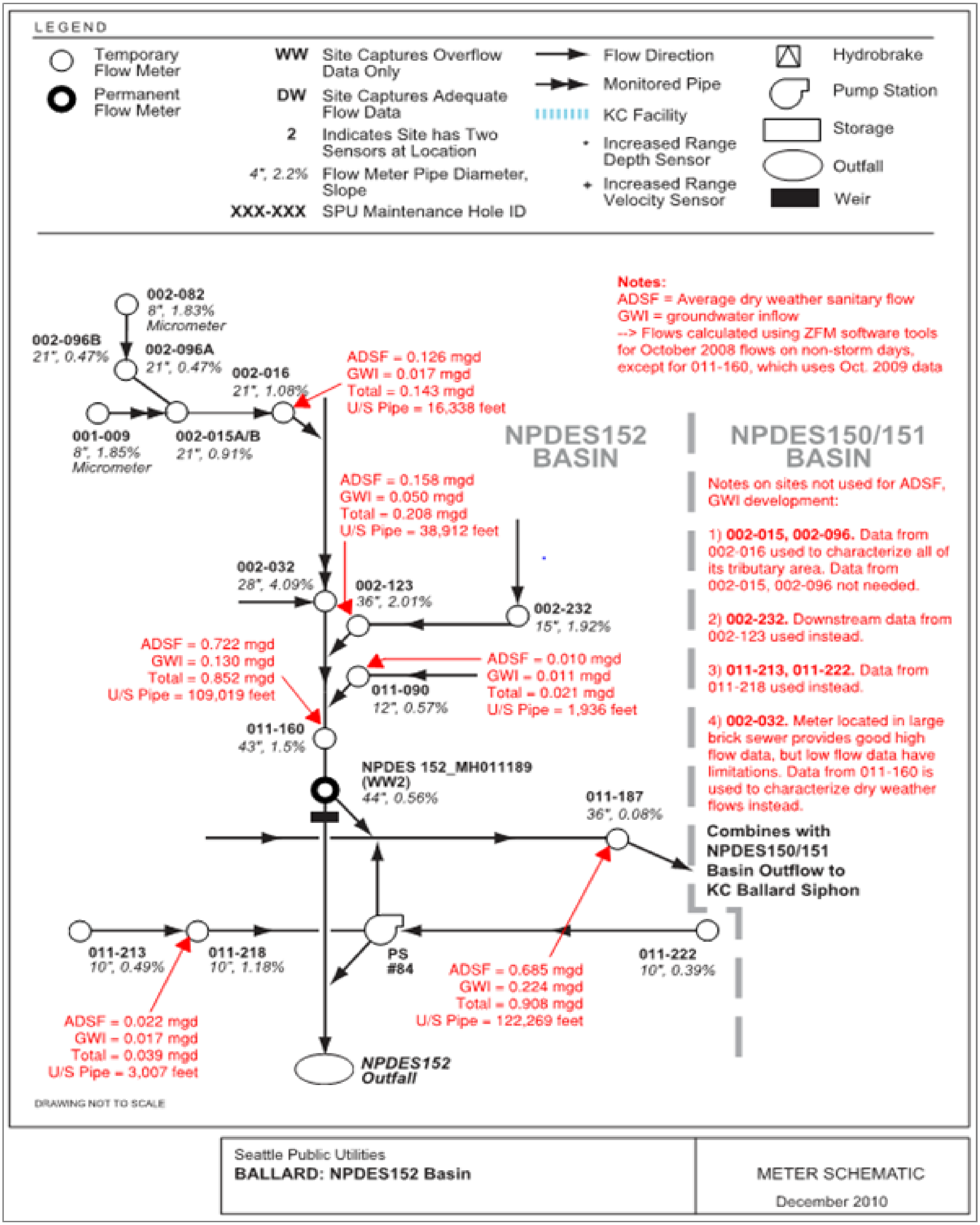


Figure 6-4. Schematic Dry-Weather Flow Summary for the Outfall 152 Basin

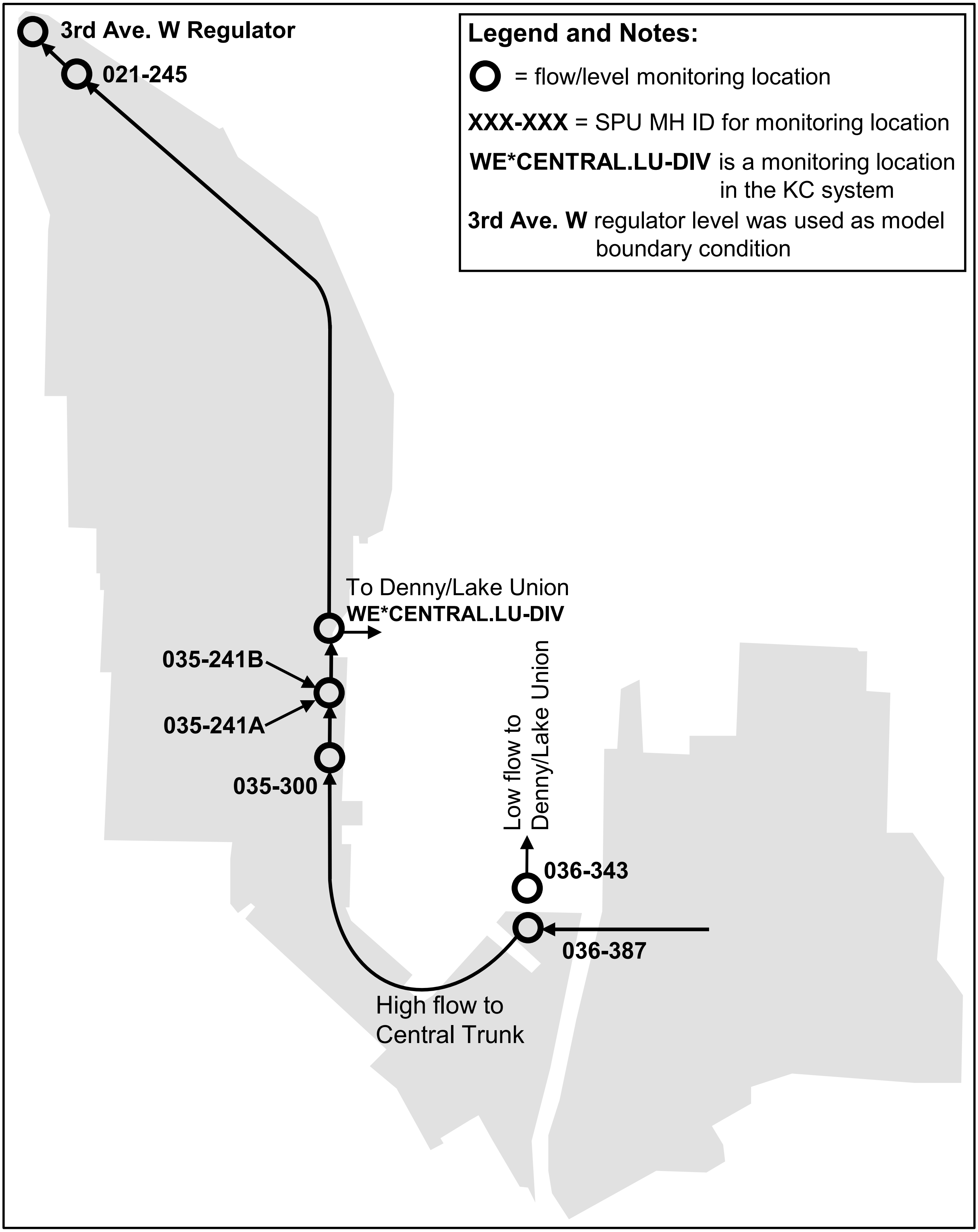
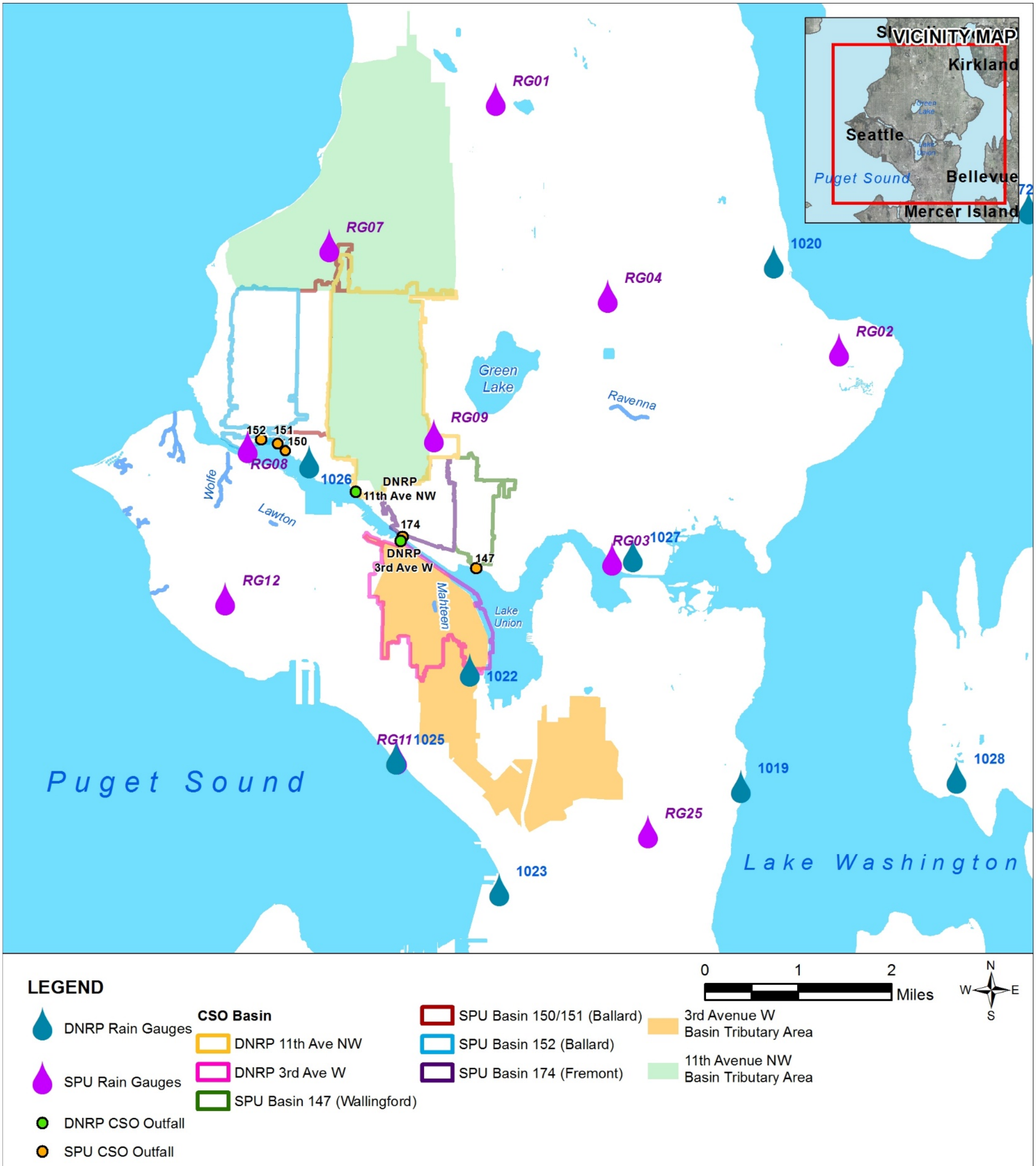


Figure 6-5. Schematic Flow Monitoring Summary for the 3rd Avenue W CSO Basin



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Final Facility Plan

Project Vicinity Rain Gauges

January 2018 | **Figure 6-6**

10 Recommended Option

This chapter provides additional engineering and environmental information for the Ship Canal WQ Project. Chapter 9 provided some information regarding the project. This chapter fully describes the project and presents O&M requirements developed after the recommended option was selected by SPU and DNRP. The detailed configuration presented in this chapter will be subject to additions, modifications, or deletions of described facilities during final design as project understanding and performance requirements are refined and additional data are collected.

10.1 Overview

The Ship Canal WQ Project will provide offline storage of combined wastewater in a deep storage tunnel constructed between the Ballard and Wallingford CSO areas, on the north side of the Ship Canal. The project will control SPU's Ballard CSO basins (Outfalls 150,151, and 152), SPU's Fremont CSO basin (Outfall 174), SPU's Wallingford CSO basin (Outfall 147), DNRP's 3rd Avenue W Overflow Structure (DSN008), and DNRP's 11th Avenue NW Overflow Structure (DSN004). Figure 1-1 in Chapter 1 provides a general project area overview and general Ship Canal WQ Project configuration. Figures 10-1 and 10-2 provide a more detailed plan view of the Ship Canal WQ Project facilities location and main system components.

The main components of the Ship Canal WQ Project include the storage tunnel and appurtenances, flow diversion and conveyance facilities to divert and convey SPU and DNRP CSO flows into the tunnel, and a pump station and effluent conveyance to drain the tunnel back into the wastewater system for secondary treatment at West Point Treatment Plant. The shared storage tunnel and appurtenances identified during conceptual planning will include the following:

- Storage tunnel will have an 18-foot 10-inch nominal inner diameter, measuring approximately 14,000 feet long.
    - The stored combined sewage in the storage tunnel will flow from the East Portal in Wallingford westward to the TEPS in Ballard.
    - The tunnel alignment primarily will be in the street right-of-way along the north side of the Ship Canal.
- Seven diversion structures will divert combined sewage away from existing CSO outfalls to the tunnel.

- Five drop structures will convey combined sewage from the surface into the storage tunnel; four structures will have odor control systems. Additional odor control facilities will be evaluated during final design.
- A pump station (TEPS facility) will be located at the West Portal and have an average capacity of 32 MGD.

Conveyance facilities will include the following:

- Gravity sewer line to convey flows from SPU's diversion structure at Ballard Outfalls 151 (approximately 300 linear feet of 48-inch-diameter pipe) and 152 (approximately 2,000 linear feet of up to 60-inch-diameter pipe) to the tunnel drop shaft.
- Gravity sewer line to convey flows from SPU's diversion structure at Fremont Outfall 174 to the tunnel drop shaft (approximately 200 linear feet of 30-inch to 36-inch-diameter pipe).
- Gravity sewer line to convey flows from DNRP's diversion structure at 3rd Avenue W (under the Ship Canal) to the tunnel drop shaft (approximately 740 linear feet of 18-inch to 60-inch-diameter pipe).
- Gravity sewer line to convey flows from DNRP's diversion structure at 11th Avenue NW to the tunnel drop shaft (approximately 120 linear feet of 60-inch to 72-inch-diameter pipe).
- Gravity sewer line to convey flows from SPU's diversion structure at Wallingford Outfall 147 to the tunnel drop shaft (approximately 1,000 linear feet of 36-inch-diameter pipe).
- Effluent discharge piping to convey flows from the tunnel pump station to SPU's local sewer (approximately 100 linear feet of 24-inch-diameter pipe) and DNRP's existing Ballard Siphon wet-weather barrel forebay (approximately 1,900 linear feet of 48-inch -diameter pipe).

All conveyance sizing and quantities, including the storage tunnel, are approximate estimates based on current design to date. Actual diameters, lengths, and alignments of conveyance facilities will be determined during final design phase.

Gravity sewer lines to convey flows from SPU's diversion structures at Ballard Outfalls 151 and 152 and Wallingford Outfall 147 to the tunnel drop shafts have been excluded from the cost share in accordance with the Joint King County/Seattle CSO Initiative Work Plan Item 4: Cost-Sharing Method for Joint Capital Projects (SPU and King County, 2012). These conveyance lines are the sole responsibility of SPU.

Control strategies for system operation will be refined during final design. Following are the minimum control volumes:

- SPU Basins:
  - Fremont (Basin 174): 1.56 MG
  - Wallingford (Basin 147): 2.36 MG
  - Ballard (Basin 152): 6.55 MG
  - Ballard (Basin 150/151): 0.58 MG
- DNRP Basin:
  - 3rd Avenue W (DSN008): 4.07 MG
  - 11th Avenue NW (DSN004): 3.14 MG

Each agency has calculated the control volumes required to meet their independent needs. Although calculation methods vary between the agencies, SPU and DNRP agree that these are the minimum control volumes to be provided by the Ship Canal WQ Project.

Following are key system components of the recommended option:

- **Storage Tunnel—**The storage tunnel will have an 18-foot 10-inch nominal inner diameter. The tunnel will have a depth of 50 to 80 feet for most of its alignment, depending on the alignment revisions during the project final design. Flows will enter the storage tunnel by gravity and be pumped to the local SPU sewer and DNRP regional interceptor when downstream capacity in these systems is available. A flushing system at the East Portal will be used to clean the storage tunnel following operation to remove accumulated solids and debris.
- **TEPS—** A pump station with an average capacity of 32-MGD will be constructed at the West Portal, located within and above the deep shaft used to construct the tunnel to access the tunnel effluent for pumping. An above-grade building will provide secured access to the pump station dry-well and wet-well areas. An on-site diesel-powered generator will provide standby power. The TEPS will be designed for automated operation (unstaffed) and include safety and ventilation systems; electrical and control systems; access considerations and spatial considerations for on-site maintenance; permanent lifting equipment; and other operational systems required for safe long-term O&M activities.
- **Drop Shafts, Portals, and Vortex Drop Structures—**Drop shafts and portals will be finished facilities located along the tunnel alignment providing conveyance functions and tunnel access. Located within the West Portal (wet well), 11th Avenue NW Drop Shaft, North 3rd Avenue/174 Drop Shaft, South 3rd Avenue Drop Shaft, and East Portal, vortex drop pipes will convey overflows vertically downward from near-surface conveyance pipelines to the storage tunnel and allow movement of air to the odor control facilities. The drop shafts and portals will also provide access to the

tunnel along the alignment for entry into the tunnel by staff as appropriate. Standby diesel-powered generators located at the portals and most drop shafts will provide sufficient backup power to control systems communications equipment, instrumentation, and nearby control gates located at conveyance system diversion structures.

- **Conveyance—**This project will include structures needed to intercept combined sewer flows during storm events from the SPU and DNRP CSO basins. Gravity pipelines will convey flows to the storage tunnel. Diversion structures with control gates will direct water either into the tunnel or to existing outfalls. Conveyance elements will also include the TEPS effluent discharge pipeline that will convey pumped flows to the Ballard Regulator Station and may include a new grit removal structure in SPU's Basin 152 upstream of the CSO interception structures. The primary anticipated construction method for conveyance pipes will be open-cut construction. Some sections will be constructed using a trenchless method (microtunnel) to avoid extended surface impacts; cross under critical utilities, railroads, and streets; and construct the 3rd Avenue W CSO connection under the Ship Canal to the North 3rd Avenue/174 Drop Shaft. Real-time controls, including automated adjustable gates, and level and flow sensors will be included at diversion structures to determine flows diverted to the storage tunnel and the existing outfalls.
- **Odor Control—**An odor control system incorporating a fan and activated carbon-scrubbing media to treat foul air from the tunnel will be located at the TEPS. An underground electrical and mechanical vault containing activated-carbon odor control system, mechanical, electrical, and control systems will be located at the 11th Avenue and North 3rd Avenue/174 Drop Shafts and the East Portal. Odor control will be provided to South 3rd Avenue from the North 3rd Avenue/174 odor control system. Odor control at other locations will be evaluated during final design.
- **Modifications to Existing System—**Existing structures may be modified based on the results of hydraulic modeling completed during final design.

In addition to the key project components described above, the project will incorporate the following elements:

- **24th Avenue NW Pedestrian Pier Improvements—**A considerable portion of tunnel construction spoils and other waste materials will be transported to a disposal site using barges. The existing 24th Avenue NW Pedestrian Pier located adjacent to the West Portal will require reconstruction in its current location to accept the anticipated loading equipment required for the effective use of barges. When the project is completed, the reconstructed pier will be converted back to a public amenity.
- **Outfall 151 Rehabilitation—** The existing 18-inch-diameter wood-stave Outfall 151 is in poor condition, and rehabilitating it during Ship Canal Project construction would

be less disruptive to the community than rehabilitating it in a separate construction project. SPU plans to replace both the existing Outfall 151 and the existing 30-inch-diameter Outfall 150 with a single 48-inch-diameter outfall. This replacement Outfall 151 will be installed under the new 24th Avenue NW Pedestrian Pier.

10.2 Layout

10.2.1 Proposed Facilities

The detailed configuration of proposed facilities presented in this section will be subject to additions, modifications, or deletions during final design as project understanding and performance requirements are refined and additional data is collected.

10.2.1.1 Storage Tunnel

The new storage tunnel alignment starts at the upstream East Portal located on City-owned property at the northeast corner of N 35th Street and Interlake Avenue N. The alignment follows N 35th Street west in the right-of-way to Fremont Avenue N and continues along Fremont Place N and N 36th Street. Near the intersection of Leary Way NW and N 36th Street, the alignment connects to the North 3rd Avenue/174 Drop Shaft (in the right-of-way) and completes a turn northwards along Leary Way NW. The alignment continues northwards along Leary Way NW to NW 45th Street and completes a turn westward on NW 45th Street. Near 11th Avenue NW, the tunnel connects to the 11th Avenue Drop Shaft and continues west in the right-of-way along NW 45th Street. Near 15th Avenue NW, the alignment shifts northwest and follows Shilshole Avenue NW in the right-of-way until reaching the West Portal located on City-owned property at the southeastern corner of Shilshole Avenue NW and 24th Avenue NW.

The tunnel alignment includes a “tunnel easement envelope” that provides a horizontal and vertical offset to protect the tunnel from future surface and subsurface development. This envelope generally extends 20 feet from the top/bottom, and 10 feet from the lateral sides of the tunnel. Permanent easements for the tunnel envelope will be negotiated with private property owners where the envelope limits fall outside of public right-of-way.

The alignment generally follows paved arterial or secondary streets and attempts to avoid residential street right-of-ways. These routing criteria were developed to reduce impacts to private property from a tunnel machine intervention should this be required during construction.

10.2.1.2 West Portal Site

The West Portal site is located on 2.15 acres of City-owned property at the southeastern corner of Shilshole Avenue NW and 24th Avenue NW. This site is bound to the north by a rail spur line (operated by the Ballard Terminal Rail Road Company), to the west by 24th Avenue NW, to the south by Salmon Bay, and to the east by an adjacent private parcel containing parking lots and commercial/industrial buildings. The West Portal site consists primarily of paved parking with some vegetated planting strips and buffers. A former restaurant is located at the southern end at the Salmon Bay waters edge. The 24th Avenue NW Pedestrian Pier is located at the site's southwest corner. The site is generally graded flat with some grade changes supported by retaining walls and rockery walls. Primary tunnel construction activities and the permanent TEPS location will be at the West Portal site.

10.2.1.3 11th Avenue Drop Shaft Site

The 11th Avenue Drop Shaft site is located in the public right-of-way along NW 45th Street between 11th Avenue NW and 9th Avenue NW. The proposed site layout is shown on Figures 10-13 and 10-14. A portion of the site currently extends onto private property to the south. This area is needed to construct the tunnel drop shaft and subterranean access corridor. A buried electrical and mechanical vault will be located adjacent to the drop shaft structure in the right-of-way. A new diversion structure (downstream of the existing overflow structure) to convey flows to the tunnel or outfall will be constructed in the right-of-way on DNRP's 11th Avenue NW outfall pipeline near the intersection of 11th Avenue NW and NW 45th Street.

10.2.1.4 North 3rd Avenue/174 Drop Shaft Site

The North 3rd Avenue/174 Drop Shaft site is located in the public right-of-way along NW 36th Street between 3rd Avenue NW and Leary Way NW. The proposed site layout is shown on Figures 10-17 and 10-18. A portion of the site currently extends onto King County-owned and SDOT properties to the south. The King County-owned parcel is the location of the forebay for the new Fremont Siphon crossing for the North Interceptor. This area is needed for constructing the tunnel drop shaft and housing a permanent buried electrical and mechanical vault. SPU will work with DNRP to ensure existing facilities will not be impacted by the construction and to obtain necessary temporary and permanent easements.

10.2.1.5 South 3rd Avenue Drop Shaft Site

The South 3rd Avenue Drop Shaft site is located at the West Ewing Park parking lot east of the terminus of 3rd Avenue W at the Ship Canal in the right-of-way. The paved parking lot is generally graded flat and is currently owned by the City. The proposed site layout is shown on Figure 10-21. This area will be used to construct the permanent drop shaft connection that will convey flows from the 3rd Avenue W diversion to a new pipe (microtunnel) under the Ship Canal. This microtunnel will connect to the North 3rd Avenue/174 Drop Shaft. A new outfall diversion structure will be constructed on DNRP's

3rd Avenue W outfall pipeline near the intersection of 3rd Avenue W and W Ewing Street, south of the Ship Canal Trail.

10.2.1.6 East Portal Site

The East Portal site is located at 3500 Interlake Avenue N. This property is owned by the City (Finance and Administrative Services [FAS]). The proposed site layout is shown on Figure 10-24. All permanent structures associated with the tunnel East portal, including a small above-grade electrical building, will be located on the site. The site generally slopes downward to the south, with retaining walls supporting the eastern and northern boundaries. A building on this site was recently demolished and the site has been converted to a parking lot. An agreement is being finalized to lease the site during construction and purchase the required property for the completed facility. Excess property would be retained under FAS ownership.

10.2.2 Revisions to Existing Facilities and Site Access

SPU will close some existing facilities and site access to the public throughout the construction duration. At the West Portal site, access to the 24th Avenue NW Pedestrian Pier will be closed during project construction as the pier is rehabilitated and used for loading tunnel excavation spoils onto barges with conveyors. The parking lot near the South 3rd Avenue Drop Shaft will be closed during construction for work and contractor staging.

Portions of the existing Burke-Gilman Trail will be temporarily closed and rerouted around the North 3rd Avenue/174 Drop Shaft and 11th Avenue Drop Shaft sites when constructing the deep tunnel shafts and connecting conveyance pipelines to the drop structures. Temporary lane closures will also be required as part of constructing the North 3rd Avenue/174 Drop Shaft site.

Access to the new CSO facilities by maintenance vehicles will be from the right-of-way onto City- or King County-owned properties or directly in the right-of-way. Dedicated parking spaces will be provided on City or King County-owned properties at the West Portal, 11th Avenue Drop Shaft, North 3rd Avenue/174 Drop Shaft, and East Portal. Parking spaces at the North 3rd Avenue/174 Drop Shaft will be marked with parking hour restriction signage marked for Class C (SPU and Seattle Department of Parks and Recreation) vehicles.

Measures to prohibit entry to the construction area without proper authority will include a temporary fence. Replanting of existing surface areas disturbed by construction activities and not covered by new features or pavement will consist of native plantings, shrubs, and trees in accordance with the Seattle Department of Parks and Recreation recommendations for site improvements to the Burke-Gilman Trail or SDOT recommendations for right-of-way improvements. Replanting activities on sites owned by the City or King County will be designed by SPU's landscaping consultant during final design.

Constructing conveyance pipelines will temporarily restrict access to some driveways and parking. SPU will work with DNRP to determine the feasibility of using DNRP-owned properties for parking during construction. If determined feasible, SPU will obtain the required temporary construction easements.

10.2.3 Access to Proposed Facilities

Access to the tunnel portals and electrical and mechanical vaults will be via hatches (rated for HS-25 loading) at the ground surface. Other areas of these structures will contain removable lifting slabs for less frequent maintenance activities. These facilities and access points will be generally located outside of vehicular travel lanes.

10.2.4 Street Frontage Right-of-Way Improvements

Street frontage right-of-way improvements are not anticipated for this project and will be confirmed based on the requirements of the SDOT and Seattle Department of Construction and Inspections.

10.2.5 Stormwater

The Ship Canal WQ Project consists of improvements that also are classified as "parcel based" with stormwater requirements, described in Seattle Municipal Code 22.805.050. The West Portal site is in a separated storm drain area, discharging storm flows from the site to Salmon Bay. This option proposes to construct the new TEPS facility with approximately 43,580 square feet of replaced impervious surface (most of which is considered pollution generating). A total of 63,000 square feet of impervious surface currently exists at the site. This option will remove approximately 19,650 square feet of impervious surface and replace with landscaping and planting areas. Therefore, according to the 2016 Director's Rules for the City's Stormwater Code (Seattle Municipal Code Chapters 22.800-22.808), runoff from the site triggers water quality treatment and onsite stormwater management. To estimate treatment requirements, pollution-generating impervious surfaces include driveway and parking lots, while non-pollution-generating surfaces include concrete sidewalks.

This option also will implement onsite stormwater management (Seattle Municipal Code 22.805.020.F), which may include runoff reduction methods of permeable pavement and amended soils. The project will incorporate bioretention planters at the West Portal site for water quality treatment.

Design elements to treat and convey stormwater will be revised as appropriate as the project design and construction management strategy is developed in the future. Runoff generated from right-of-way surfaces qualifies for an exemption from these standards since the entire project is to improve overall water quality.

10.2.6 Landscaping

Existing landscaping at the different project sites and along the near-surface conveyance alignments will be removed to limits required to complete construction. Most landscaping in the public right-of-way along the deep storage tunnel alignment (outside of indicated drop shaft sites) will not be directly impacted as part of the tunnel construction because the tunnel will be constructed using a subsurface tunnel boring machine. However, landscaping removal (primarily tree pruning or limbing) may be limited during construction to install and periodically monitor settlement monitoring equipment. The project will strive to preserve outstanding trees.

Project site landscaping will vary by location. The West Portal and East Portal sites will be landscaped using a mix of native plants and preferred decorative species. This project aspect will be finalized during final design. The 11th Avenue and North 3rd Avenue/174 Drop Shaft sites are primarily in the right-of-way. Landscaping will be as prescribed by current SDOT street planting requirements. Similarly, landscaping along conveyance alignments will be as prescribed by current SDOT street planting requirements.

10.2.7 Hydraulic Profiles

Existing and proposed structures and conveyance pipelines are shown with hydraulic profiles for the peak-flow operating conditions anticipated for the recommended option on Figures 10-3 through 10-9. These profiles schematically represent the interconnections of the proposed project components and connections to the existing SPU and DNRP wastewater conveyance systems. Hydraulic profiles may change based on overall system refinements made during final design.

10.3 Storage Tunnel

The storage tunnel will be located under primarily public right-of-way north of the Ship Canal. The nominal 18-foot 10-inch finished inner diameter storage tunnel will extend from Ballard to Wallingford, and will be approximately 14,000 feet long. The storage tunnel will store excess combined sewer flows from SPU Basins 147, 150/151, 152, and 174. The storage tunnel will also store excess combined sewer flows from DNRP Basins 3rd Avenue W (DSN008) and 11th Avenue NW (DSN004).

During storm events, flows from any of the six basins will be piped to the storage tunnel via dedicated conveyance pipes from diversion structures and enter the storage tunnel via drop shafts and portals located at each end of the tunnel and at two locations along the alignment. Flows entering the storage tunnel will be stopped by motor actuated gates at each diversion once a pre-determined level in the storage tunnel has been reached. Once a gate has closed, excess flows will be routed to that CSO basin's associated outfall. The system will be provided with motor-actuated gates, and controls will be provided to allow flexibility to effectively control the system to meet performance standards.

A self-cleaning system using a control gate located at the eastern-most upstream end (East Portal) will provide a flushing wave (approximately 40,000 gallons of stored sewage) to move settled materials from the storage tunnel to the downstream western-most end (West Portal). Modeling analysis was used to confirm the volume required for the flushing wave to achieve a minimum velocity of 3 feet per second along the entire tunnel alignment. This velocity value was selected based on the typical grain-size distribution of sediment typically found in domestic combined sewer systems and the ability of a flushing wave at that velocity to resuspend materials and convey them to the terminal end of the tunnel. A pump station at the West Portal (TEPS) will pump the materials and flushing water to the Ballard Regulator Station near the ground surface. The Ballard Regulator Station discharges to the DNRP system and flow is conveyed to the West Point Wastewater Treatment Plant for treatment.

The storage tunnel will be kept under a slight (approximate 0.1-inch water column) negative air pressure by continuously drawing air from the storage tunnel headspace and treating it with an odor control system at the West Portal. Odor control is included at each of the other three portals to treat foul air during tunnel filling.

For the basis of determining a project envelope of construction and environmental impacts and costs, the tunnel turning radii and construction shaft sizing is based on an 18-foot 10-inch finished inner diameter tunnel.

Access to the storage tunnel will be through the tunnel portals and drop shaft structures. The design includes ladders and platforms for inspection and maintenance activities. Access to the ladders will be through surface hatches or buried corridors leading to the portal or drop shaft. Removable concrete panels at the portals can be lifted by crane to facilitate placing equipment into the storage tunnel, such as a small skid steer or other machinery used for cleaning or repairs.

10.4 Tunnel Access Locations

Portals and drop shafts are finished facilities located along the tunnel alignment that provide conveyance functions and tunnel access. Access locations are located at the

West Portal (wet well of the TEPS), 11th Avenue Drop Shaft, North 3rd Avenue/174 Drop Shaft and East Portal sites. Portals and drop shafts range in depth from approximately 60 feet to 100 feet (to bottom of tremie slab), and an inner diameter from 10 feet to 50 feet. Drop structures within the access structures convey flows vertically downward from near-surface conveyance pipelines to the storage tunnel below. The access locations provide entrance into the tunnel along the alignment for entry by staff as needed to perform maintenance. Standby diesel-powered generators situated above ground are located at each portal to provide backup power to instrumentation and nearby control gates located at conveyance system diversion structures.

A fifth deep shaft structure, the South 3rd Avenue Drop Shaft, will be located south of the Ship Canal in the West Ewing Mini Park parking lot east of 3rd Avenue W and W Ewing Street to convey flows from the 3rd Avenue W outfall to a new microtunnel connection to the North 3rd Avenue/174 Drop Shaft. The South 3rd Avenue Drop Shaft will have a 20-foot inner diameter and will be approximately 80 feet deep.

10.4.1 West Portal

The West Portal is adjacent and connected to the TEPS facility, and serves as the TEPS wet well and a point of access to the tunnel. Refer to Section 10.5 for additional detail of the West Portal configuration. Figure 10-10 shows the proposed site plan of the West Portal and TEPS site. Figures 10-11 and 10-12 show three-dimensional views of the proposed finished TEPS facility, constructed inside of the West Portal structure that will be used for tunnel construction before being reconfigured as the final TEPS facility.

10.4.2 11th Avenue Drop Shaft

The 11th Avenue Drop Shaft site is located in the public right-of-way along NW 45th Street between 11th Avenue NW and 9th Avenue NW. Figures 10-13 and 10-14 show the proposed site plan of the 11th Avenue NW Drop Shaft. Figures 10-15 and 10-16 show three-dimensional views of the proposed structure. The finished drop shaft will send flows to the tunnel through an adit, which is a short-tunneled connection to the main tunnel. A second small shaft drilled directly over the tunnel will provide ventilation and odor control. Primary access to the mechanical/ electrical vault is via a surface hatch in the planting strip/sidewalk south of the drop shaft in the right-of-way. This access hatch provides access to the buried facilities without requiring crews to temporarily close NW 45th Street. A vault containing metering equipment may be placed in-line with the conveyance past the diversion and before the drop structure.

The electrical and mechanical vault at this site is located east of the drop shaft in the right-of-way. The standby diesel-powered generator is located above grade in close proximity to the electrical and mechanical vault. Buried odor ductwork from the electrical and mechanical vault connects to the smaller secondary drop shaft east of the primary drop shaft.

The 11th Avenue NW connection pipeline will enter the drop shaft from the west and connect to an approximately 60-foot deep drop pipe to vertically convey flows to the storage tunnel. The drop pipe will discharge to a concrete stilling well offline from the main tunnel alignment in the bottom of the drop shaft before entering the tunnel through the adit.

10.4.3 North 3rd Avenue/174 Drop Shaft

The North 3rd Avenue/174 Drop Shaft is located in the public right-of-way along NW 36th Street between 3rd Avenue NW Leary Way NW. Figures 10-17 and 10-18 show the proposed site plan for the North 3rd Avenue/174 Drop Shaft. Figures 10-19 and 10-20 show three-dimensional views of the proposed structure. The finished drop shaft will be directly accessible from the surface through hatches, lift slabs, and maintenance hole openings in the structure lid located in the NW 36th Street right-of-way. Primary access to the drop shaft will be through hatches in the right-of-way, requiring crews to temporarily close NW 36th Street for inspection and maintenance. A caged ladder assembly extends from the access hatches to the bottom of the drop shaft.

The odor control system at this site will be located in the electrical and mechanical vault, near the new Fremont Siphon and in SDOT right-of-way. The standby diesel-powered generator is located above grade in close proximity to the electrical and mechanical vault on SDOT property. Buried odor ductwork from the electrical and mechanical vault connects to the upper part of the drop shaft. SPU will work with DNRP to ensure no conflicts occur to existing King County facilities and will obtain required necessary temporary and permanent easements.

The Outfall 174 connection pipeline enters the portal structure from the east and connects to a drop pipe that vertically conveys flows to the storage tunnel. The drop pipe will be up to 30 inches in diameter and affixed to the portal wall. A vault containing metering equipment may be placed in-line with the conveyance past the diversion and before the drop structure. The drop pipe will discharge to a concrete stilling well offline from the main tunnel alignment in the bottom of the portal.

The 3rd Avenue W microtunnel connection pipeline enters the North 3rd Avenue/174 Drop Shaft from the southeast and directly discharges to the same concrete stilling well that accepts flows from the CSO 174 drop pipe. Flows will cascade into the tunnel opening via an adit, which is a short-tunneled connection to the main tunnel.

10.4.4 South 3rd Avenue Drop Shaft

The South 3rd Avenue Drop Shaft site is located at a parking lot east of the terminus of 3rd Avenue W at the Ship Canal in the right-of-way. Figure 10-21 shows the proposed site plan for the South 3rd Avenue Drop Shaft. Figures 10-22 and 10-23 show three-dimensional views of the proposed structure. The finished drop shaft will be directly accessible from the surface through hatches, lift slabs, and maintenance hole openings in the structure lid located in the parking lot. Primary access to the structure will be through hatches in the parking lot, requiring crews to temporarily restrict use of the parking lot for inspection and maintenance.

The 3rd Avenue W connection pipeline will enter the drop shaft structure from the south. The drop pipe will be up to 60-inch inner diameter and affixed to the drop shaft wall. The drop pipe discharges to a bottom of the shaft and flows enter two gravity conveyance pipes (42-inch and 18-inch), sized for different flows and constructed inside of a 94.5-inch-diameter microtunnel that conveys flows under the Ship Canal to the North 3rd Avenue/174 Drop Shaft.

Odor control and standby power will be provided by the North 3rd Avenue/174 facility north of the Ship Canal. Odor control will be performed with an air jumper pipe, and back-up power will be provided by conduits, both located within the microtunnel.

10.4.5 East Portal

The East Portal site is located at 3500 Interlake Avenue N. Figure 10-24 shows the proposed site plan for the East Portal. Figures 10-25 and 10-26 show three-dimensional views for the proposed structure. The finished portal will be directly accessible from the surface through hatches, lift slabs, and maintenance hole openings in the structure lid located in the City property. Primary access to the structure will be through hatches in the driveway of the finished site.

The odor control facility at this site is located in a vault attached to the portal shaft. The standby diesel-powered generator is located above ground on the south side of the portal on the City property. A small above-grade electrical building will also be located near the portal shaft.

The Outfall 147 connection pipeline enters the portal structure from the south and connects to a drop pipe that vertically conveys flows to the storage tunnel. The drop pipe will be up to 36 inches in diameter and affixed to the portal wall. The drop pipe will be held in place by supports anchored to the wall and concrete encased to protect the pipe material from corrosion and damage from maintenance activities and provide additional structural support. The drop pipe will discharge to a concrete stilling well offline from the main tunnel alignment in the bottom of the portal. This stilling well will also serve to temporarily hold back flow for release by a control gate. When the control gate releases a flushing wave, the stored CSO will flow into the tunnel to remove sediment and carry it to the TEPS wet well.

10.5 Tunnel Effluent Pump Station

A pump station with average pumping capacity of 32-MGD will be constructed at the West Portal. The primary purpose of the TEPS is to dewater the storage tunnel once capacity is available in the downstream conveyance system. TEPS will house both a primary tunnel dewatering pumping system and a secondary tunnel dewatering system.

The primary tunnel dewatering system will use two duty pumps and one standby pump. The primary tunnel dewatering pumps are designed for raw sewage service, dry-pit submersible-type pumps and are identical in size. The rated capacity for each pump at the design condition is 16 MGD at 60 feet total dynamic head. Primary tunnel dewatering pumps will be equipped with variable speed drives to pump a range of flows based on the downstream sewer capacity at the DNRP North Interceptor.

Initial tunnel dewatering will start with secondary wet-well dewatering pumps. Two of the secondary wet-well dewatering pumps will operate before starting the primary tunnel dewatering pumps to remove the majority of settled material in the wet well. The primary tunnel dewatering pumps will start to drain the tunnel and continue pumping until the level in the wet well is below a shut-off point. Once the tunnel is emptied to the level where the primary pumps shut off, the secondary wet-well dewatering pumps will continue to drain the wet well. The secondary wet-well dewatering pumps will also drain the wet well following tunnel self-cleaning with the flushing wave generated at the East Portal.

The TEPS facility will be located within and above the deep shaft used to construct the tunnel. An above-grade building will provide secured access to the pump station dry-well and wet-well areas. The proposed TEPS site plan and sections are shown on Figures 10-10 and 10-11.

Odor control at the TEPS will be located in the TEPS building. Odors will be mitigated using activated carbon media housed in the odor control vessel to scrub odor-causing compounds from air drawn from the tunnel and wet well. Corrosion-resistant ductwork connecting the odor control structure to the TEPS wet well will be buried underground. The odor control fan will be located inside of the TEPS building to provide better noise mitigation from continuous fan operations. Scrubbed air will discharge from the odor control fan through an exhaust stack through the roof of the TEPS building.

An on-site diesel-powered generator will provide standby power for up to 48 hours of continuous operation for the pump station equipment during power outages and will be housed in a sound-reducing cover system to minimize noise impacts. The TEPS will be designed for automated operation (unstaffed) and include safety and ventilation systems; electrical/control systems; access considerations, including stairways and an elevator; spatial considerations for on-site maintenance; permanent lifting equipment; and other operational systems required for safe long-term O&M activities.

The TEPS effluent discharge pipeline will consist of 100 linear feet of 24-inch-diameter pipe and approximately 1,900 linear feet of 48-inch-diameter pipe. The effluent discharge pipeline will begin at the north side of the TEPS and extend northeast to the north side of Shilshole Avenue NW. The proposed alignment generally follows Shilshole Avenue NW southeast to the Ballard Regulator Station, located on the corner of Shilshole Avenue NW and NW Dock Pl. The effluent discharge pipeline will be constructed using open cut construction. Figure 10-4 shows the hydraulic profile of the TEPS effluent discharge pipeline under anticipated operating conditions.

The effluent discharge piping will connect to the Ballard Regulator Station at the existing siphon drop connection. Further design coordination with DNRP will be required for the connection at this location.

10.6 Auxiliary Portal and Drop Shaft Facilities

The 11th Avenue Drop Shaft, North 3rd Avenue/174 Drop Shaft and East Portal will have auxiliary structures and equipment required for O&M. An underground electrical and mechanical vault at these portal sites will contain an odor control system, mechanical equipment, electrical equipment, and control panels to modulate nearby control gates. The electrical and mechanical vaults will be constructed as separate structures nearby or adjacent to the portal structures. Access to the electrical and mechanical vaults will be through hatches and stairways to grade level. The exterior dimensions and configurations of the electrical and mechanical vault vary by site. The typical electrical and mechanical vault will be buried to minimize impact to the use of the sites and right-of-ways after construction. The design of the access hatches to the vault will address utility conflicts and rerouting, maximize maintenance access, and minimize visual presence of the hatches at the surface.

The odor control system will consist primarily of a carbon adsorption scrubber vessel, grease filter, and exhaust duct. Provisions for a future fan and an in-line duct silencer include reserved space and connection points to the carbon vessel and ductwork. The system will allow foul air vented from the tunnel during filling to pass through the carbon media for treatment before discharge to the environment. The odor control system will connect to the portal structure with buried, corrosion-resistant ductwork or piping. Up to 200 feet of buried ductwork is anticipated for each of these facilities. Treated-air discharge ductwork will extend from the vault to exhaust plenums at the ground surface nearby.

Wash down water for cleaning the electrical and mechanical vault interior will be provided for maintenance. A small air gap tank (designed to meet WAC 246-290-490, Orange Book G2.2.2.3 G-1 and H-3 [Ecology, 2008], and Table 6.3 of Uniform Plumbing Code) and service pump system will be installed in the electrical and mechanical vault in the same space as the odor control system. Water service connections to the electrical and mechanical vaults from nearby water mains will be detailed during final design.

SPU provides on-site standby power for projects that are considered critical infrastructure and where significant consequences could occur if continuous power was lost (for example, a sewage pump station). The modulating gates in diversion structures are critical to managing CSO event flows in the project area. Loss of power will prevent the gates from closing or opening during an event. However, this will not prevent the sewer collection system from continuing to operate. An on-site dedicated standby diesel-powered generator will be located above grade at the West Portal, 11th Avenue NW, North and South 3rd Avenue drop shafts, and East Portal sites since the storage tunnel is expected to be used 40 to 60 times per year.

10.7 Basin 150/151 Conveyance

The proposed Basin 150/151 conveyance pipe alignment extends down 24th Avenue NW from existing MH 011-233 to the CSO 150/151 diversion structure located on the northwest corner of the West Tunnel Portal site. Overflows from the existing CSO weir structure will be diverted from the outfall pipe and conveyed through the new diversion structure to the tunnel. Approximately 300 feet of 48-inch-diameter conveyance pipe will be used to convey overflows from the outfall pipe diversion point to the tunnel. The new outfall pipe from the diversion structure to a new maintenance hole on the existing outfall will be approximately 140 feet of 48-inch-diameter conveyance pipe. The peak conveyance rate from Basin 150/151 used for sizing pipelines is approximately 60 MGD.

This diversion structure will be a standard pre-cast 12-foot-diameter maintenance hole modified to control flows into the tunnel. A sluice gate mounted on a concrete support wall will be raised and lowered by an electric gate actuator located above ground. An adjustable weir will separate the tunnel flow channel from the outfall pipe; when the water level rises above the weir a CSO event will occur. A removable baffle for floatables may be located in the outfall chamber. The baffle would prevent floatables from discharging into the Ship Canal during CSO events.

Access to the diversion structure will be provided via a standard maintenance hole located above the access bench on the tunnel flow side of the structure. The electric gate actuator will be located above ground, allowing routine maintenance without entry into the structure. A security fence will be constructed at the site to protect the gate actuator from vandalism. Backup power to equipment associated with the replacement Outfall 151 will be provided by the diesel-powered generator at the TEPS and connected by buried electrical conduit from the TEPS to the specific equipment locations.

10.8 Basin 152 Conveyance

The proposed Basin 152 diversion structure is located on 28th Avenue NW, south of NW 56th Street. The rectangular cast-in-place structure will have three channels to direct flow into three conveyance routes. Combined sewer flows will be intercepted from the existing sewer system and flow to the diversion structure upstream (north) of an existing maintenance hole. Dry-weather flows will pass through the diversion structure and continue to the existing wastewater system. Approximately 300 feet of 48-inch-diameter conveyance pipe will connect the interception point to the reconnection point for dry-weather flows. The peak conveyance rate from Basin 152 used for sizing pipelines is approximately 80 MGD.

Flows will overtop the first weir and be directed to the tunnel. Tunnel conveyance will start at the diversion structure and extend east along NW 56th Street, turning south on south at 24th Avenue NW and continues to the TEPS at the West Portal. Approximately 2,000 feet of up to 60-inch-diameter conveyance pipe constructed in up to 84-inch-diameter microtunnel will extend from the Outfall 152 diversion structure to a new maintenance hole near the West Portal. The final 200 feet of conveyance to the tunnel drop structure at the TEPS will be constructed using open cut construction. The alignment, pipe sizes, and construction methods will be further refined during the design phase.

When the tunnel has reached its storage capacity, the gate actuator downstream of the diversion structure will close the sluice gate. The water level will rise to the second weir and flow into the third channel of the diversion structure, which will be connected directly to the existing outfall pipe. Flows entering the third channel will cause a CSO event to occur.

Backup power to equipment associated with Outfall 152 will be provided by the diesel-powered generator at the TEPS and connected by buried electrical conduit from the TEPS to the specific equipment locations.

A grit removal structure may be constructed near 28th Avenue NW between NW 56th Street and NW 57th Street. The structure would be located in the right-of-way and would require approximately 3,000 square feet of construction area. Grit removal requirements in the collection system will continue to be evaluated during final design.

10.9 11th Avenue NW Conveyance

Overflows from the 11th Avenue NW Overflow Structure located at 11th Avenue NW and NW 45th Street will be directed to the 11th Avenue NW Drop Shaft through the proposed diversion structure located on the northeast corner of the intersection. Approximately 50 feet of 72-inch-diameter and 70 feet of 60-inch-diameter conveyance will connect the existing CSO structure to the diversion and drop shaft. The peak conveyance rate from the 11th Avenue NW CSO basin used for sizing pipelines is approximately 168 MGD.

The proposed rectangular cast-in-place concrete diversion structure will have a single channel to allow overflows from the existing DNRP CSO structure to pass directly into the tunnel. Sluice gates will be used to control or stop flow into the tunnel. Gate actuators could be hydraulic or electric-type, as determined in final design.

When the tunnel is at capacity, the diversion structure sluice gate will close, allowing the water level to rise and overtop the weir, causing a CSO event to occur. Flows will exit the structure via approximately 10 feet of 72-inch-diameter conveyance pipe connecting to the existing outfall line approximately 100 feet south of the existing 11th Avenue NW CSO Overflow Structure.

10.10 3rd Avenue W Conveyance

Overflows from the existing DNRP 3rd Avenue W Overflow Structure will be diverted from the existing outfall pipe downstream from the overflow structure. The beginning of the proposed diversion will include a drop maintenance hole located in 3rd Avenue W. Flows will drop through this maintenance hole and be directed to a new diversion structure located in a parking area adjacent to the Ship Canal trail on property currently owned by the City. The conveyance alignment continues to the proposed South 3rd Avenue Drop Shaft in the parking lot of the West Ewing Street Mini Park. Approximately 90 feet of 60-inch-diameter conveyance pipe will connect the new diversion structure to the drop shaft. Flows will enter the drop shaft and continue to the South 3rd Avenue W/174 Drop Shaft through approximately 650 feet of 18-inch and 42-inch-diameter conveyance pipe installed in a 94.5-inch-diameter microtunnel constructed under the Ship Canal. The peak conveyance rate from the 3rd Avenue CSO basin used for sizing pipelines is approximately 103 MGD.

The proposed cast-in-place concrete diversion structure will have a single channel allowing overflows from the 3rd Avenue W CSO Overflow Structure to pass directly into the tunnel. Gate actuators could be hydraulic or electric-type, as determined in final design.

Sluice gates will be used to control or stop flow into the tunnel. When the tunnel capacity is reached, the new diversion structure sluice gate will close, allowing the structure to fill with water until overtopping an adjustable weir. Water will enter the outfall chamber and exit the structure via the existing outfall pipe, causing a CSO event to occur.

10.11 Basin 174 Conveyance

A new concrete diversion structure will be located south of an existing diversion structure on 2nd Avenue NW. This diversion structure will convey flows to the King County siphon during normal conditions. During overflow conditions, the diversion structure will convey water to pass to the tunnel. When the tunnel has reached storage capacity, the gate actuator closes the sluice gate and the structure fills until water overtops the second weir and a CSO event occurs.

The outfall pipe alignment from the Outfall 174 overflow structure will extend south along 2nd Avenue NW and connect to a new maintenance hole planned as part of the Fremont Siphon Replacement project. Approximately 120 feet of 36-inch to 48-inch-diameter conveyance pipe will connect the Outfall 174 diversion structure to the outfall. The peak conveyance rate from Basin 174 used for sizing pipelines is approximately 20 MGD.

The proposed Outfall 174 diversion structures will be accessed via maintenance holes with weir walls and flow channels installed. Normal flows will pass directly through the structure and continue to treatment. Overflows will overtop the weir and be directed to the Outfall 174 diversion structure and on to the tunnel/outfall conveyance.

Gate actuators could be hydraulic- or electric-type, as determined in final design.

10.12 Basin 147 Conveyance and Pipe Storage

Basin 147 basin is divided into two subbasins with separate conveyance: 147A and 147B. For Subbasin 147A, a new concrete diversion structure will be located west of an existing diversion structure at the intersection of Stone Way N and N 34th Street. This diversion structure will convey flows to the King County siphon during normal conditions. During overflow conditions, the diversion structure will convey water to pass to the tunnel. When the tunnel has reached storage capacity, the gate actuator closes the sluice gate, and the structure fills until water overtops the second weir and a CSO event occurs. For Subbasin 147B, a new concrete diversion will be located at the existing diversion structure located on Woodlawn Park Avenue N and N 34th Street. This diversion structure will convey flows to the King County siphon during normal conditions. During overflow conditions, the diversion structure will convey water to pass to a new diversion structure and then to the tunnel. When the tunnel has reached storage capacity, the gate actuator closes the sluice gate, and the structure fills until water reaches the elevated pipe in the new diversion structure and a CSO event occurs.

The proposed conveyance system starts at the new Subbasin 147A diversion structure located west of the existing diversion structure at the intersection of Stone Way N and N 34th Street. Overflows from Subbasin 147A will overtop the weir in the new diversion structure and join Subbasin 147B overflows and be directed to the tunnel. The conveyance alignment will follow Stone Way N north to the intersection of N 35th Street before continuing east on N 35th Street to the East Portal. Approximately 90 feet of 30-inch-diameter and 670 feet of 36-inch-diameter conveyance pipe would compose the total Basin 147 conveyance pipelines. The peak Basin 147 conveyance rate used for sizing pipelines is approximately 30 MGD.

When the tunnel has reached storage capacity, the gate actuator at the East Portal will close the sluice gate. The water level will rise to the elevated pipe in the new Subbasin 147A diversion structure and be connected directly to the existing outfall pipe. Flows entering the elevated pipe will cause a CSO event to occur.

Back-up power to equipment associated with Outfall 147 will be provided by the diesel-powered generator at the East Portal and connected by buried electrical conduit from the East Portal to the specific equipment locations.

10.13 Operational Modes

Six operational modes are identified by SPU and DNRP as part of the Ship Canal WQ Project Facility Plan development. These modes are described in detail below with specific steps and operational activities. The intent of the operational modes described herein is to operate the storage tunnel system by relying on an automated network of gates, instruments and controls with direct operator supervision and interagency communication and cooperation (including data-sharing) to meet the regulatory requirements for CSO reduction for the targeted SPU and DNRP CSO basins.

DNRP and SPU are developing an O&M plan in accordance with the signed JPA. The operational modes described in this section will be further refined in the final O&M plan. Additional control modes that will be evaluated and refined during the final design phase include modes for tunnel inspection and full storage (tunnel has reached storage capacity but not draining).

10.13.1 Mode 1: Tunnel Filling

During tunnel filling, automated gates at secondary diversion (interceptor) structures will be in their opened position, allowing flows to enter the tunnel. As water levels rise in the combined sewer system, primary weirs at existing DNRP overflow diversion structures and new SPU diversion structures will overtop with combined sewer flows. Flows will enter the storage tunnel at each of the portal locations through the new diversion conveyance systems and the storage tunnel will begin to fill. Instruments at each interceptor structure will monitor level/flow to determine flow from each location into the tunnel.

Gates will actuate to stop flows to the tunnel based on the final operating strategy, which may limit inflows based on the storage level in the tunnel, the storage volume allocation for each basin, and/or a rate-of-rise threshold. A secondary level monitoring and control system at the TEPS wet well will provide an “all stop” water elevation set point and will also close the gates once the water in the wet well reaches that elevation. When gates are in the closed position, this will cause the overflow weirs at diversion structures to overtop, sending combined sewer flow to existing outfalls. If rain continues, combined sewer flows will discharge from existing CSO outfalls. During final design, SPU and DNRP will further develop the operational strategy for tunnel filling to maximize the use of the tunnel storage capacity.

10.13.2 Mode 2: Tunnel Draining

During tunnel draining, water level instruments in DNRP’s North Interceptor (location to be determined during final design, near DNRP’s Fort Lawton Tunnel, upstream of West Point) will indicate that the North Interceptor can accept flows from the storage tunnel without creating negative conditions at the West Point Treatment Plant and any intermediary DNRP facilities. Additional monitoring of DNRP facilities, including the Ballard Regulator Station, may be required to further define the pumping limits during tunnel draining. These requirements will be refined during final design.

10.13.3 Mode 3: Tunnel Cleaning

Tunnel cleaning will begin once the wet well is dewatered and capacity is available in the downstream DNRP system (including the Ballard Regulator Station and North Interceptor). Instruments measuring the wet-well level will provide a signal to the system control center and the control gate at the East Portal will open. The self-cleaning system using the control gate will open, providing a flushing wave (approximately 40,000 gallons of stored sewage) to move settled materials from the storage tunnel to the downstream West Portal. Modeling analysis was used to confirm the volume required for the flushing wave to achieve a minimum velocity of 3 feet per second along the entire tunnel alignment. This velocity value was selected based on the typical grain-size distribution of sediment typically found in domestic combined sewer systems and the ability of a flushing wave at that velocity to resuspend materials and convey them to the terminal end of the tunnel. A pump station at the West Portal (TEPS) will pump the materials and flushing water to the Ballard Regulator Station near the ground surface. The Ballard Regulator Station discharges to the DNRP system, and flow is conveyed to the West Point Treatment Plant for treatment.

10.13.4 Mode 4: Standby Mode

In standby mode, the system will be ready to accept flows from the combined sewer basins. All of the motor-actuated gates at the interceptor structures will be in the open position. The tunnel may experience infiltration through joints or cracks over time during standby mode. The secondary wet-well dewatering pumps will pump groundwater that infiltrates into the tunnel to the local SPU sewer once a predetermined water elevation in the wet well has been reached.

10.13.5 Mode 5: Continuous Operation (Filling/Draining)/System Optimization

Under continuous operation, the storage tunnel and TEPS will receive continuous data from the DNRP North Interceptor level instrument and flow/volume information from each of the combined sewer basin diversion structures. This mode will be further evaluated in final design to determine a strategy that addresses back-to-back storm events and how tunnel draining must be stopped or proceed at a reduced pumping rate.

10.13.6 Mode 6: Manual Control

Under manual control mode, the operator will modify system controls from automated to manual control mode. The operator could selectively open and close control gates and adjust the duration and pumping rate of the TEPS pumping systems. The SCADA system interface will provide the operator with applicable level information to help control the system to prevent overflows. The control set points will continue to generate alarms when the storage tunnel approaches and reaches its fill level and when flows overtop weirs. Staff will implement appropriate control actions for the following situations:

- Power failure and restoration
- Communications failure and restoration
- Programmable logic controller self-diagnostics alarms and restoration
- Level and flow measure calibration, out of range (high and low), and restoration
- Set point entry range checking

10.14 Sizing

Hydraulic modeling provided the basis for the estimated volume required for storage tunnel and sizing of the conveyance system. Chapter 6 describes the hydraulic modeling. Table 10-1 summarizes important hydraulic conditions and design flow rates for both the existing system and the system after the proposed changes. Table 10-2 provides major project dimensions and sizes. The values presented in these tables will be updated during final design.

Table 10-1. Design Flows and Hydraulic Conditions	
System Operating Parameter	**Approximate Value**
TEPS average pumping rate	32 MGD
Approximate storage volume for storage tunnel	29 MG
Basin 152 peak conveyance flow rate	80 MGD
Basin 150/151 peak conveyance flow rate	60 MGD
11th Avenue NW CSO peak conveyance flow rate	168 MGD
Basin 174 peak conveyance flow rate	20 MGD
3rd Avenue W CSO peak conveyance flow rate	103 MGD
Basin 147 peak conveyance flow rate	30 MGD
Approximate flushing wave volume	40,000 gallons

Table 10-2. Sizing of Ship Canal Water Quality Project Facilities		
Dimension	**Approximate Value**	**Unit**
Storage Tunnel		
Approximate tunnel storage volume	29	MG
Tunnel length	14,000	Feet
Tunnel nominal inner diameter	18.83	Feet
Maximum depth of cover to tunnel crown	Up to 80	Feet
Tunnel Slope	0.25	Percent
West Portal		
Depth (to finished floor for tunneling) – dry-well shaft	Up to 100	Feet
Inner diameter	85	Feet
11th Avenue Drop Shaft		
Depth (to finished floor for tunneling	Up to 100	Feet
Inner diameter	Up to 15	Feet
Odor control flow rate	7,000	cfm
Odor control shaft inner diameter	Up to 8	Feet
North 3rd Avenue/174 Drop Shaft		
Depth (to finished floor for tunneling	Up to 100	Feet
Inner diameter	Up to 32	Feet
Odor control flow rate	12,000	cfm
South 3rd Avenue Drop Shaft		
Depth (to finished floor for tunneling)	Up to 100	Feet
Inner diameter	Up to 25	Feet
East Portal		
Depth (to finished floor for tunneling)	Up to 100	Feet
Inner diameter	Up to 35	Feet
Odor control flow rate	4,000	cfm

Table 10-2. Sizing of Ship Canal Water Quality Project Facilities		
Dimension	**Approximate Value**	**Unit**
Flushing volume storage	40,000	Gal
Tunnel Effluent Pump Station		
Design pump flow rate – primary pump system	16	MGD
Total dynamic head at design point	60	Feet
Number of pumps – primary pump system	2	Each
Pumping rate at design point	32	MGD
Design pump flow rate – secondary pump system	2.0	MGD
Number of Pumps – secondary pump system	3	Each
Pumping rate at design point	2	MGD
Total dynamic head at design point	150	Feet
Odor control flow rate (active)	10,000	cfm
Standby diesel-powered generator capacity at TEPS	Up to 1.5	MW
Standby diesel-powered generator capacity at other locations	up to 100	kW
Footprint of pump station facilities	Up to 72 x 170	Feet

cfm *cubic feet per minute (air flow)*
MW *megawatt*

10.15 Design Life

The basis of design assumes the storage tunnel has a 100-year design life and the primary equipment has a 25-year design life. Routine maintenance of the facility and replacement of equipment will occur as needed to realize the design life.

10.16 Solids Management

The design of the proposed storage tunnel will include a control gate and sewage reservoir for flushing and self-cleaning at the East Portal so that solids will not accumulate in the storage tunnel. However, the tunnel design allows for access and cleaning through the TEPS wet well and portal/drop shaft structures if necessary to remove additional materials.

The solids management practices for the sewer lines leading to and from the proposed storage tunnel are the same as those SPU currently implements elsewhere in the sewer system. These practices entail accessing the sewer lines via maintenance holes and using a Vactor™ truck to extract solids. The solids are routinely taken to the SPU Haller Lake facility and decanted. The City's solid waste contractor then disposes of the remaining solids.

10.17 Ability to Provide Additional Control Volume

In the unlikely event additional control volume is needed, it could be achieved by one or more of the following options:

- Performing infiltration reduction measures
- Implementing the Residential RainWise Program
- Constructing additional storage in the Ship Canal WQ Project basins

10.17.1 Infiltration Reduction

Due to the age of the collection system in the project area, many pipe segments are likely experiencing infiltration; however, as was found in the *Pilot Project Report: Regional Infiltration and Inflow Control Program* (King County, 2004), the majority of the infiltration is likely occurring in smaller diameter lateral and side sewers on private property. Even when City workers identify sources of infiltration, such as leaking pipe joints, quantifying the flow rate of groundwater that leaks into the sewer during wet weather can be extremely difficult. Infiltration reduction projects are unreliable in achieving specified reductions of flow required for CSO control because predicting or measuring the anticipated or achieved reduction level can be difficult.

Other combined sewer agencies across the nation, including others in the Northwest, consider infiltration reduction a good asset management practice but do not rely upon the reduction of flow to achieve CSO reduction requirements. SPU frequently performs the types of construction typically associated with infiltration reduction, such as cured-in-place pipe lining, to protect the structural integrity of the pipeline or remove obstructions such as roots. Infiltration reduction is usually a secondary benefit of rehabilitating the pipe. SPU considers reducing infiltration an ongoing effort to maintain a reduced level of combined sewer flows. Any such reduction in the combined sewer flows helps ensure the facility has adequate capacity.

10.17.2 Residential RainWise Program

SPU's Residential RainWise Program could also reduce combined sewer flows within the basin. The program aims at reducing the amount of stormwater runoff from private properties into the storm drainage system and sewer collection system. Removing residential stormwater connections from the combined system reduces the volume and flow rate of wet-weather peak flows.

10.17.3 Construct Additional Storage

If the storage tunnel was determined to need additional capacity, supplemental storage could be added by constructing a connecting tunnel or tank storage in the project area, depending on the storage volume needed. Additional analysis will be required to determine the preferred option.

10.18 Estimated Operations and Maintenance Staffing Needs

The O&M strategic direction of the recommended option is as follows:

- Design the system to "Keep It Simple" for O&M.
- Design tunnel and pipes to be maintainable from ground surface whenever possible.
- Provide for entry and maintenance.
- Monitor the system during operations to indicate when proactive maintenance is required to ensure the system functions properly.

Staff will perform regular maintenance to meet the design life of the facility and ensure proper operation, including required instrument calibration. Table 10-3 shows the types of O&M activities that could occur, the frequency of each activity, and staffing requirements to perform those activities. The list includes normal inspection and maintenance activities. Minor repairs, cleaning, adjustments, and needed replacement of minor components will be part of those activities. Major repair or replacement of structures, equipment, and systems are not included. A future O&M Plan to be developed by both agencies per the JPA will supersede the assumptions shown.

10.19 Design Parameters

10.19.1 Site Selection

Site selection was initiated as part of the SPU LTCP (Volume 2 of the *Plan to Protect Seattle's Waterways*; SPU, 2015a). Sites for the tunnel portals and CSO diversion structures were identified as part of this process. This facility plan refined the locations to those described and shown herein. Conveyance alignments were generally identified as well as part of the SPU LTCP (SPU, 2015a) and refined as part of this facility plan.

10.19.2 Site Design

The finished site design for sites inside and outside of the right-of-way must provide adequate access, working space, and parking for maintenance of the system. Minimizing impact to existing on-site and adjacent land uses is an important project site design parameter.

Table 10-3 Routine Operation and Maintenance Activities for Ship Canal Water Quality Project Tunnel

Frequency	Typical Activities	Equipment and Staff	Impacted Area
Quarterly	Inspect differential pressure indicated by the gauges on the mist/grease filter and carbon beds in odor control units; Replace and clean the fouled filter pads as needed.	Service truck and 2 staff	Right-of-way areas where access to buried structures requires lane closures Outside activities at the TEPS facility will generate noise to the adjacent parcels for the duration of work.
	Exercise valves, motor-operated gate, and pumps in facilities vault.		
	Inspect debris build-up on walls and weirs and clean as necessary.		
	Inspect landscaping and maintain grounds at TEPS.		
	Calibrate instruments at required locations based on schedule.		
	Inspect and maintain indicator lights, displays, pressure gauges, and monitoring equipment.		
Annually	Inspect mechanical and electrical equipment for wear and corrosion.	Service truck and 2 staff	Right-of-way areas where access to buried structures requires lane closures Activities for the annual maintenance could be performed in conjunction with quarterly inspection activities (requires an additional service truck and 2 staff). Outside activities at the TEPS facility will generate noise to the adjacent parcels for the duration of work.
	Inspect and maintain heating-ventilation-air-conditioning supply/exhaust, and odor control fans.		
	Collect carbon sample or use the carbon sampling probe to check the carbon media life.		
	Remove and inspect secondary wet-well dewatering drain pumps (offsite servicing).		
	Test and certify backflow prevention device.		
	Test standby diesel-powered generator system.		
	Replace carbon media.		
	Remove and inspect dry-well sump pumps (offsite servicing).		
Annually	Clean conveyance piping for connected overflow diversion pipes at each portal and drop shaft location.	Service truck, Vactor™ truck, and 4 staff	Right-of-way areas where access to buried structures requires lane closures

Table 10-3 Routine Operation and Maintenance Activities for Ship Canal Water Quality Project Tunnel

Frequency	Typical Activities	Equipment and Staff	Impacted Area
	Clean effluent discharge piping.	Service truck, Vactor™ truck, and 4 staff	Right-of-way areas where access to buried structures requires lane closures
Infrequent (every 5 to 25 years)	Replace mechanical equipment (valves, pumps, piping).	Service truck, Vactor™ truck, and 4 staff	Right-of-way areas where access to buried structures requires lane closures
	Inspect microtunnel.	Specialty Equipment or subcontractor	
	Inspect structure of storage tunnel and drop shafts/portals.	Service truck and 2 staff	Right-of-way areas where access to buried structures requires lane closures
	Inspect flow-monitoring vault and equipment.	Service truck and 2 staff	Right-of-way areas where access to buried structures requires lane closures
Very Infrequent (every 25+ years)	Replace large equipment at TEPS.	Service truck and 3 staff	Inside TEPS Facility
	Perform seismic Inspections.	Crane, semi-truck, service van, 3 to 10 staff	Right-of-way areas where access to buried structures requires lane closures
	Repair tunnel, portal, drop shafts, and diversion structures.	2 Service trucks and 6 to 10 staff	Right-of-way areas where access to buried structures requires lane closures

10.19.3 Construction/Earthwork

Shoring for earthwork will be of a type appropriate for the available space on the site or in the right-of-way and other site conditions. Shoring for earthwork must adequately support the sides of the excavation and protect adjacent areas and structures.

10.19.4 Structural/Geotechnical

Additional geotechnical borings were completed between February and September 2016 along the tunnel alignment and at key facility locations and are currently being analyzed.

The tunnel will be constructed in a mixture of very dense or hard glacially overconsolidated glacial till (gravel, sand, and silt), outwash (sand and gravel), and interglacial fluvial (sand and gravel) and lacustrine deposits (silt and clay). Groundwater pressures along the tunnel invert will be between 3.5 to 5 bars, depending on final tunnel depth. The access shafts will be constructed through similar soils, but will also encounter looser and softer soils near the ground surface. The potential for liquefaction and lateral spreading exists at three of the shaft locations. Additional analysis will be required during the final design to better define the risk and need for mitigation.

Pressurized-face tunneling methods, along with gasketed segmental liners, will be required to resist groundwater and soil pressures. The shaft excavations will likely require relatively tight shoring with dewatering, excavation in the wet, and tremie slabs, or ground improvement, to provide a stable excavation base.

10.19.5 Stormwater

Stormwater design will follow the City's Stormwater Code for onsite stormwater management water quality treatment of runoff and flow control where required. The design flow rate is the rate at or below which 91 percent of the total runoff volume for the simulation period is treated (Seattle Municipal Code 22.805.090.B1). The stormwater design also will incorporate onsite stormwater management, including the use of amended soils, permeable surfacing, bioretention planters, or some combination of these elements. This option may include runoff reduction methods of permeable pavement and amended soils. These improvements will infiltrate direct precipitation, remove pollutants, reduce runoff, and reduce the size of future drainage facilities. Additional site-specific soils analysis is required as part of evaluating and selecting onsite stormwater management strategies.

The option will incorporate the following design approaches. A detailed assessment of the drainage systems in the project basins will be completed as part of the final design.

10.19.5.1 West Portal Site

Runoff from the West Portal site generally flows south towards Salmon Bay. The existing site stormwater system will be demolished during construction. The portion of the site that will accommodate the TEPS facility will be paved or graded to direct runoff to on-site water quality treatment facilities including filter planter boxes, bioswales or other treatment technologies. Other onsite stormwater management strategies for the parcel could consist of a porous sidewalk/driveway surfaces around the facility. These surfaces will infiltrate direct precipitation, reduce runoff, and reduce the size of future drainage facilities. Runoff from the West Portal site will be discharged to the Ship Canal through SPU's rehabilitated Outfall 151.

10.19.5.2 11th Avenue Drop Shaft Site

Runoff from the 11th Avenue Drop Shaft site will remain in the existing right-of-way by using grading and curb and gutter to direct flows to existing drainage structures. Onsite stormwater management strategies for the site could consist of a porous sidewalk in the disturbed area or adding treatment systems to treat runoff.

10.19.5.3 North 3rd Avenue/174 Drop Shaft Site

Runoff from the North 3rd Avenue/174 Drop Shaft site will remain in the existing right-of-way by using grading and curb and gutter to direct flows to existing drainage structures. Onsite stormwater management strategies for the site could consist of a porous sidewalk in the disturbed area or adding treatment systems to treat runoff.

10.19.5.4 East Portal Site

Runoff from the East Portal site generally flows south and eastward towards the north end of Lake Union. The existing site stormwater system will be demolished during construction. The site will be paved or graded to direct runoff to on-site water quality treatment facilities including filter planter boxes, bioswales or other treatment technologies. Other onsite stormwater management strategies for the parcel could consist of a porous sidewalk/driveway surfaces around the facility.

10.19.6 Architecture and Landscaping

At the West Portal site, an above-grade building is proposed. Design elements such as exhaust stacks, intake and exhaust units, and other exposed above-grade features will be designed to be compatible with the existing site's aesthetic characteristics. At the East Portal site, a small above-grade building is also being proposed. Design elements will be reviewed with stakeholders and will blend with the neighborhood architectural fabric.

10.19.7 Operations and Maintenance and Facility Inspection Considerations

An important design objective is for simple and reliable operation and low maintenance. This includes avoiding the need to enter the storage tunnel to perform regularly scheduled O&M activities by including a post-event solids removal system. The storage tunnel flushing system will have automated operation. Scheduled maintenance will require entry into the East Portal to inspect the flushing system control gate.

The design will allow access for personnel and equipment to enter the storage tunnel and portals. For example, the design will incorporate removable concrete panels to allow large equipment to be placed inside and removed from the storage tunnel via the portals. Access hatches for scheduled O&M activities will have fall protection grating. Staff will develop additional O&M procedures for the tunnel, portals and flushing system as needed during final design. The TEPS wet well, storage tunnel, and drop shafts/portals are not planned for full occupancy, and are therefore considered confined spaces. Staff will implement confined space entry procedures before entering these structures. The TEPS dry well and electrical and mechanical vault at each of the drop shaft/portal sites are planned for full occupancy and will include appropriate life safety systems (e.g., ventilation, lighting, access provisions) to meet current code requirements.

O&M personnel will monitor the overall facility remotely during operation to verify that the mechanical, electrical and instrumentation and controls systems are working properly

The TEPS facility O&M and inspection will follow industry standard practices for pump stations and CSO facilities. O&M procedures are currently being established for odor control systems recently constructed at other CSO storage facilities. Additional O&M procedures will be developed as needed during final design and documented in the O&M plan per the JPA.

10.20 Feasibility of Implementation

Based on an evaluation of engineering, hydraulics, construction, O&M, and environmental aspects, implementation of the Ship Canal WQ Project Tunnel option appears to be feasible with no fatal flaws. DNRP will participate in the Ship Canal WQ Project based upon the approved Consent Decree modification and the signed JPA.

10.21 Environmental Impacts

SPU evaluated the Ship Canal WQ Project option as part of the *Plan to Protect Seattle's Waterways* and the 2014 Plan EIS (SPU, 2015a). To address new and modified project elements, SPU prepared a project-specific SEIS (SPU, 2017). The SEIS addresses new and modified information for the following environmental elements identified by SEPA:

- Earth and Groundwater
- Surface Water
- Air Quality and Odors
- Fisheries and Biological Resources
- Land Use and Shoreline Use and Visual Quality
- Recreation
- Transportation
- Noise and Vibration
- Energy and Climate Change
- Historic, Cultural, and Archaeological Resources

These analyses consist of review of updated information, fieldwork, and modeling.

Project impacts identified in this section will be minimized by implementing proper measures and BMPs that will be defined during final design.

10.21.1 Earth and Groundwater

Construction-related impacts to earth and groundwater would be associated with excavation, dewatering, trenching, tunneling, and the presence of contaminated soil and groundwater.

10.21.1.1 Erosion and Dewatering

Areas that are disturbed during construction will be subject to increased erosion, and erosion control measures will be required.

Dewatering may be required in some locations to prevent groundwater from interfering with construction. However, the project will be designed to require minimal amounts of dewatering. Dewatering during excavation below the groundwater table could result in settlement of nearby structures, roadways, and utilities. However, the potential for impact is considered low if proper measures to minimize and avoid dewatering are used.

10.21.1.2 Contaminated Materials

Potential for encountering contaminated soils during tunnel boring is low because the tunnel will be deep. If contaminated soil is encountered, then it will be managed in accordance with Ecology Model Toxics Control Act (MTCA) and other applicable requirements.

The contamination associated with the West Portal at the Salmon Bay Hotel Group property is documented and would require cleanup under Department of Ecology MTCA requirements. Contaminants detected in soil removed from the East Portal or other construction areas would also be removed in accordance with applicable requirements.

Property acquisition and demolition needs will be determined during final project design, including any specific management requirements under the Asbestos Hazard Emergency Response Act. All contaminated materials will be handled in accordance with applicable requirements and disposed of at an appropriate facility. Removing contaminated materials during construction would benefit human health and worker safety and reduce the risk of future contamination of earth and groundwater.

10.21.1.3 Vibration and Settlement

Based on currently available data, building damage from vibration during tunnel excavation is not anticipated because of the depth of the tunnel. As is typical of tunnel projects, the Ship Canal WQ Project will require excavation that could result in minor ground settlement in localized areas. Where needed, protective measures such as grouting will be used during tunnel boring to prevent or limit settlement. These measures have been successfully used on tunnel projects in the Seattle area. The use of these measures is expected to prevent damage to most buildings and utilities.

Ground settlement could occur in areas where soils are excavated and dewatering occurs. Construction activities, including pile driving and sheet pile installation, could cause vibration and also result in ground settlement. Excessive settlement could impact or apply loads to nearby roadways, rail lines, utilities, and structures. More detailed analysis will be conducted during project design to determine areas where soils could settle.

If areas were prone to settlement, engineers will propose measures to minimize effects. Any settlement from constructing the portals, drop shafts, or conveyance elements is expected to be minor and would be repaired either during or after construction.

10.21.1.4 Spoils Disposal

Based on current plans, an estimated maximum of approximately 409,000 cubic yards of spoils would be generated from site demolition, excavation, foundation installation, and ground improvement activities. An estimated maximum of approximately 275,000 cubic yards will be excavated during tunnel construction, and the remainder of spoils will come from the rest of the project. Spoils that are unsuitable for reuse by the Ship Canal WQ Project will need to be disposed at an appropriate facility. The disposal site will be determined during final project design, but clean soils will be hauled to a permitted approved facility for final disposal.

Potential impacts resulting from disposal of spoils include erosion and sedimentation where excavated materials are stored onsite or if they are spilled during transport. These impacts will be evaluated and mitigated during final design.

Transport of spoils by rail car, barge, and truck could result in dust deposited on roadways, rail corridors, or water. Covering of loads during hauling will reduce dust. Some of the excavated soil will originate from areas where known or suspected contamination has been identified. Soils will be tested during construction to determine if they are contaminated. If they are contaminated, they will be transported in accordance with applicable containment and transport methods to an approved disposal site.

Operational impacts on earth and groundwater resources would be minor, and removing contaminated material would benefit soil and groundwater quality.

10.21.2 Surface Water

The overall construction effects on surface water resources could include increased turbidity, increased pollutants and sediments entering stormwater runoff, and increased risk of pollutant spills. BMPs will be implemented to reduce the potential for these effects, in accordance with City of Seattle requirements. Additionally, a Stormwater Pollution Prevention Plan and a Construction Stormwater and Erosion Control Plan will be prepared to ensure that measures are in place to protect water quality, prevent erosion and sedimentation, and manage activities and potential pollutant sources.

Project operation is anticipated to result in substantial water quality benefits in the Ship Canal because the number and volume of CSO discharges will be reduced.

10.21.3 Odor and Air Quality

The Ship Canal WQ Project would cause short-term, minimal to moderate localized effects on air quality during construction activities. Construction air quality impacts adjacent to construction sites would relate to dust from disturbed soils and odors and emissions from operating heavy-duty diesel and gasoline-powered equipment, earth excavation and grading, handling and transport of excavated material, and truck trips. Use of heavy equipment and trucks would end once construction is complete, but would take place over several years in some locations. Sewer odors could also be temporarily emitted where existing sewer pipes or vaults are opened during construction. Construction BMPs would minimize impacts.

The Ship Canal Tunnel will be designed to minimize the generation of odors by using state-of-the-art odor control facilities at locations where odors could be released to the atmosphere. The project includes a system-wide odor control system equipped with automated cleaning systems and odor control systems with carbon scrubbers, mist and grease filters, and fans at some locations. Additional odor control systems, which include carbon scrubbers and filters, will be installed at the drop shafts to allow air vented from the tunnel during filling to pass through the carbon media for treatment before discharge to the environment.

10.21.4 Fisheries and Biological Resources

For most of the project, any disturbance of terrestrial habitat would occur on paved or developed areas. Vegetated areas in this urban setting are disturbed but may provide some habitat to urban wildlife. Impacts to vegetated areas would be limited and would have minimal effect, given the adaptability of wildlife living in these areas.

In-water work related to pier reconstruction, barge use, and potential outfall replacement could cause short-term, localized turbidity plumes; underwater noise and vibration; and increased underwater shading from moored work barges. Any temporary effects are not considered significant. The project area provides poor salmon habitat. While salmonids migrate through the area, the Ship Canal is unlikely to be used extensively by salmonids for holding and foraging. In Salmon Bay, near the West Portal site, the shoreline is lined with docks providing long-term and active boat moorage and there is very little riparian or upland vegetation. Adult salmonids migrate into the Ship Canal from Puget Sound through the Ballard Locks or the fish ladder at the Locks. Adult salmonids tend to migrate fairly quickly through the Ship Canal, with an average passage time of 1 to 4 days depending on species. Juvenile salmonids out-migrate through the Locks and fish ladder, but can also travel via culverts used to divert fresh water into the Locks, the smolt passage flumes, or the spillway gates.

Chinook salmon smolts usually take 1 to 4 weeks to pass through the Ship Canal whereas sockeye and coho salmon take less than one week. Adult out-migrating salmon, in particular Chinook salmon, often hold just upstream from the Locks in a cool water refuge near the saltwater drain before going through the Locks.

Nevertheless, all in-water work will be conducted during the work window approved by the Washington Department of Fish and Wildlife and BMPs will be implemented to minimize impacts to fish and other aquatic species (City of Seattle, 2013). Impacts to fish and fish habitat would be temporary and minimal because in-water work will occur when salmonid species are least likely to be present. SPU will work with affected Tribes to minimize impacts to Tribal fishing, and the timing of in-water work will be restricted to minimize impacts on tribal fishing. Tribal concerns regarding potential impacts to Tribal fishing would be addressed during the Corps of Engineers permitting process.

Impacting aquatic habitats from construction site runoff or in the unlikely event of construction equipment spills is a risk. However, impacts would be minimal by implementing required BMPs, as well as a Stormwater Pollution Prevention Plan and a Construction Stormwater and Erosion Control Plan.

After completion, the Ship Canal WQ Project will have a long-term beneficial effect on fish and other aquatic species due to less discharge of combined sewage. Stormwater runoff that enters the combined sewer system will be treated before discharge to Elliott Bay rather than discharged to the Ship Canal, and the tunnel will reduce CSOs from existing Ship Canal outfalls to no more than one per year on a 20-year moving average, thus improving water quality in the Ship Canal. Replacing the existing creosote-treated timber piles supporting the pier at 24th Avenue NW will reduce a contaminant source in the Ship Canal. The reconstructed pier will also have fewer piles than existing, and will likely include grated decking for increased light penetration to minimize impacts to fish and aquatic habitat.

10.21.5 Land and Shoreline Use and Visual Quality

Potential construction-related impacts on land and shoreline use and visual quality are associated with acquisition of property and easements, incompatibility of surrounding land uses, changes to views, and light and glare.

10.21.5.1 Acquisition of Property and Easements

Temporary and permanent easements from some private landowners will be needed to construct the project. This will include a “tunnel envelope” that provides a horizontal and vertical offset to protect the tunnel from future surface and subsurface development. This envelope will generally extend 20 feet from the top, bottom, and sides of the tunnel. Permanent easements for the tunnel envelope will be negotiated with private property owners where the envelope extends outside the public right-of-way. This routing was developed to reduce impacts to private property in the unlikely event a tunnel machine intervention should be required during construction.

Temporary construction easements will be needed from adjacent property owners for the West Portal. Depending on the final design of the 24th Avenue NW pier, several potential temporary property-related impacts could occur, including extending the reconstructed pier, displacing existing recreational and live-aboard boat moorage at the adjacent pier to the east, displacing the commercial pier use to the west, and using extra-long or double barges, protruding further into the Ship Canal waterway and potentially affecting waterway use.

A limited number of temporary construction easements will likely be required for construction activities or staging areas associated with constructing the drop shafts and conveyance located outside of public rights-of-way.

Some relocations will be required; the City will follow applicable requirements for property acquisition, compensation and relocation.

10.21.5.2 Incompatibility of Adjacent Land Uses

Use of the 24th Avenue NW pier for barging operations near the West Portal will cause conflicts with adjacent mooring piers, requiring temporary displacement or relocation of moorage. The use of tugs and barges will increase the use of the Ship Canal waterway but this increase in vessel traffic will not be significant.

Use of both rail and barges to haul materials and tunnel spoils is under consideration. Both of these options could have potential impacts and could be incompatible with recreational uses during the construction period.

10.21.5.3 Changes to Visual Character

Construction will temporarily affect visual character through short-term changes to views resulting from construction equipment and activities. Given the industrial character in the vicinity of the West Portal and pier, the temporary presence of the conveyor structure and use of large barges would not be a significant visual impact.

10.21.5.4 Light and Glare

Nighttime construction could be necessary for project components, resulting in light and glare impacts. Temporary lighting impacts during nighttime construction would be reduced by shielding light sources to block direct views from residential areas, and by aiming and shielding light sources to reduce spillover lighting from such areas as necessary.

After construction, permanent underground easements will have no material impact on the normal use and enjoyment of the affected properties. The former Yankee Diner building will remain in place to be sold or repurposed. The 24th Avenue NW pier will be reopened for public access. A portion of the East Portal site is anticipated to remain in City ownership following project completion. Permanent easements for the two intermediate drop shafts will not interfere with existing site uses or access. No significant impacts to land and shoreline uses are expected at West Ewing Mini Park after construction. The presence of drop shaft facilities will result in a dedicated use of the subsurface area and will restrict certain future uses in the surface area above the facilities. The area is currently used for parking, and redeveloping it to a different use is not planned.

The project's consistency with Seattle's Comprehensive Plan is the same as stated in the 2015 Plan EIS. The regulatory environment, specifically City of Seattle Land Use Code and SMP described in Section 4.8 of the 2015 Plan EIS, has not substantially changed. However, Ecology approved Seattle's SMP Update on June 1, 2015, and put it in effect on June 15, 2015. No substantive changes to standards applicable to utility services and utility lines in the approved SMP Update have been made compared with the version of the SMP Update that was reviewed at the time the 2015 Plan EIS was issued.

10.21.6 Recreation

Construction-related impacts can occur when construction is within or adjacent to a park or in a right-of-way.

10.21.6.1 West Portal

The existing pier at the 24th Avenue NW street end will be closed to recreational use for up to 4 years. Because other nearby public docks will remain open, and recreationists will be able to utilize alternate nearby facilities, this impact would not be significant. The proposed Threading the Needle Park project could not begin until the Ship Canal WQ Project is complete and the pier is no longer being used to convey tunnel spoils. However, no funding or schedule for implementing the Threading the Needle Park project is currently available. Therefore, constructing the Ship Canal WQ Project likely will not delay the park project.

Recreational users of the Ship Canal include paddle boarders, kayakers, and recreational boat users. They will likely notice construction noise and activity associated with pier construction and barging operations, but noise and activity levels would be consistent with the types of noise and activity that currently occur along the industrial shoreline.

10.21.6.2 Drop Shafts and Conveyance

Construction will potentially require temporary closure and rerouting of portions of the Burke-Gilman Trail during the 12- to 15-month construction period. Construction activities will need to be coordinated with the Burke-Gilman Trail Missing Link project construction.

Some construction activities will likely occur within Fremont Canal Park. The actual location of the drop shaft will be determined during final design. If located in the park, construction areas within the park will be fenced, and most of the park will remain available for recreational use.

Construction will likely occur in a portion of the paved parking lot of West Ewing Mini Park. During the approximate 6- to 9-month construction period, recreationists using West Ewing Mini Park will still have access to the park, but the construction area will be fenced. Park users will still be able to access the overlook, lawn areas, picnic tables, and benches during construction. However, park users will be aware of construction noise, dust, the high visibility of construction activities and fencing, and increased traffic on adjacent roads from construction truck trips.

Construction will likely occur adjacent to the Ship Canal Trail and recreation areas along the Ship Canal associated with the trail (including lawn areas and benches). During the approximate 6- to 9-month construction period, recreationists will still be able to access the trail. However, trail and park users will be aware of construction noise, dust, the high visibility of construction activities and fencing, and increased traffic on adjacent roads from construction truck trips.

Construction activities will likely also be located in the vicinity of athletic facilities at Seattle Pacific University. The Royal Brougham Pavilion will be within 150 feet of construction, and Wallace Athletic Field will be within 300 feet of construction. Construction activities will be visible and potentially audible from Wallace Athletic Field.

10.21.6.3 Construction in a Right-of-Way

Construction in road rights-of-way would temporarily interfere with informal recreation opportunities such as bicycle and pedestrian use. For the Ship Canal Project, drop shaft construction and associated conveyance activities could disrupt bicycle and pedestrian use on streets over the approximate 12- to 24-month construction period in each neighborhood. Due to the availability of alternate routes, this disruption would not be considered significant.

10.21.6.4 Hauling of Tunnel Spoils

An estimated maximum of 275,000 cubic yards of tunnel spoils (and approximately 70,000 cubic yards of soil excavated for shaft construction) will need to be hauled away from the West Portal site. Tunnel spoils will be hauled through a combination of three methods: barge, train, or truck. Most tunnel spoils likely will be hauled by barge or rail car. Depending on how the pier and barges are configured, the barges could encroach on navigation in the Ship Canal, impacting recreational canal use. Additionally, barges could preclude moorage at adjacent privately owned piers. Train traffic could cause periodic short access delays to the Burke-Gilman Trail and 11th Avenue NW, 14th Avenue NW, and 28th Avenue NW, as well as to the Ballard Locks. Bicyclists despite a high number of existing truck trips on the road and entering and exiting driveways already frequently use Shilshole Avenue NW. Therefore, bicycle use of Shilshole Avenue NW will likely not be disrupted by truck trips for this project. However, added truck trips could increase potential safety conflicts along Shilshole Avenue NW.

After construction, the project will reduce pollutant loading to the Ship Canal, with potential long-term benefits to water-based recreation. Operational impacts will be limited to those areas where permanent facilities associated with the Ship Canal WQ Project are located in or adjacent to parks at the West Portal location, the South 3rd Avenue drop shaft, and the North 3rd Avenue Drop Shaft. The 24th Avenue NW pier will be reopened for public access. The new concrete pier will have a modern design for pedestrian use and boat tie-off.

10.21.7 Transportation

Most transportation impacts would be construction-related, including disruption to vehicular and non-motorized traffic at roadways, sidewalks and trails where construction occurs, displacement of parking, and potential increases in vehicular traffic generated by construction activities. Transportation impacts during construction would include temporary roadway lane and sidewalk narrowings or closures adjacent construction activities. Some closures could require temporary detours of vehicular, transit, or non-motorized traffic.

If Ballard Conveyance is constructed via NW 54th Street, transportation impacts would be considered significant and unavoidable unless measures could be implemented to maintain adequate access to adjacent businesses during construction.

Construction-generated truck trips likely will not significantly affect roadway operations, but likely will be noticeable. Use of barge or rail to support construction activities where feasible would reduce truck trips.

Increases in train traffic during construction may require measures to minimize the potential conflict with other vehicular or non-motorized traffic.

Measures to reduce or eliminate potential construction impacts include general measures to avoid or reduce vehicle queues and delay near construction activity, maintaining vehicular and non-motorized access along roadways disrupted by construction, as well as to adjacent businesses and residences, coordinating with agencies with jurisdiction over the transportation facilities, and coordinating with affected community members.

When constructed, the Ship Canal WQ Project facilities will be located mostly underground and physically separated from transportation infrastructure and services. A small number of operational trips will be generated to support O&M.

10.21.8 Noise and Vibration

Noise generated by construction equipment and activities could impact residential areas and sensitive receptors. Operational noise impacts would be generated by pump stations, odor control facilities, maintenance, and other noise-generating equipment associated with permanent facilities.

Multiple projects, public and private, will be under construction concurrent with the Ship Canal WQ Project. Potential impacts from construction noise will depend upon the type of construction activity on a given day, the equipment used, the distance between construction activities and the nearest sensitive land use, and the existing ambient sound levels near the receptor.

Residential areas near Ballard Conveyance and Wallingford Conveyance have the greatest potential for experiencing intermittent noise impacts.

Vibration impacts such as minor cosmetic damage to structures or annoyance of occupants may occur during concrete demolition and shaft construction.

Once construction has been completed, a pump station will operate at the West Portal and an odor control systems will operate at the drop shaft locations and the East Portal. Diesel-powered generators at each of the portal and drop shaft locations will be tested for 1 hour each month.

Completed facilities operations must comply with Seattle Municipal Code sound level limits at adjacent property lines. Seattle Municipal Code 25.08.530 exempts sounds generated by emergency equipment and applies to diesel-powered generator testing as long as reasonable noise mitigation is used.

After project completion, vibration impacts are not anticipated. Equipment installed at the pump station, drop shafts, and portal locations are not anticipated to generate vibration levels high enough to cause impacts at nearby receptors.

Constructing the Ship Canal WQ Project may require nighttime construction activities at the West Portal; therefore, a nighttime noise variance may be required from Seattle Department of Construction and Inspections. Because of the project magnitude, a Major Public Project Construction Noise Variance will most likely be required. In coordination with Seattle Department of Construction and Inspections, measures to reduce the impact of noise will be developed and specified in the noise variance. To reduce construction noise at nearby receptors, measures could be incorporated into construction plans, specifications, and variance requirements. Final measures will be determined as part of permitting during final design. Additional measures could reduce operational noise impacts and may be required to meet Seattle Municipal Code sound level limits and worker safety requirements after the project has been completed. Daytime construction activities are not expected to exceed daytime sound level limits at any project sites.

To reduce vibration impacts produced during construction and operation activities, additional measures could be implemented and will be determined as part of permitting conditions established during final design.

10.21.9 Energy and Climate Change

Constructing the Ship Canal WQ Project will produce greenhouse gases, which contribute to climate change. Greenhouse gas production would primarily be associated with emissions from construction equipment and commuter vehicles, as well as embodied energy. “Embodied energy” is the energy necessary for the entire product lifecycle beginning with raw material extraction and ending with deconstruction or decomposition.

During the 6- to 7-year construction period, diesel-fueled construction equipment will require an estimated 812,608 gallons of diesel fuel. Construction worker personal vehicles will consume an estimated 640,000 gallons of gasoline. The total greenhouse gas emissions from consumption of fuels during project construction will be approximately 9,786 metric tons of carbon dioxide equivalent (CO_2e). The embodied energy required for the project will add approximately 18,841 metric tons of CO_2e. Together, the total greenhouse gas emissions during construction will be an estimated approximately 35,692 metric tons of CO_2e. This impact is considered to be minor considering the total CO_2e emissions in Seattle in 2012 were 3,728,000 metric tons of CO_2e (City of Seattle, 2014). Therefore, constructing the Ship Canal WQ Project will contribute less than 1 percent of Seattle’s annual total greenhouse gas emissions.

An estimated 35,873,760 kilowatt hours of electricity will be required to operate the tunnel boring machine, tunnel lighting and fans, yard lighting, and other construction equipment. This electricity use will be spread across the 2-year construction period of the tunnel, and the daily electric use will be a small percentage of the overall energy consumption in the region. Therefore, the impact would not be significant. O&M staff vehicles will produce minor greenhouse gas emissions. The associated annual greenhouse gas emissions are an estimated 32 metric tons.

Operating the Ship Canal WQ Project will also use electric power to run pumps and ventilation equipment. Operating the equipment could be energy intensive, but the equipment will operate infrequently, only during and after storm events. The anticipated annual electricity consumed will be approximately 2 million kilowatt hours, an amount not considered significant when compared to energy use in the City of Seattle as a whole.

DNRP's West Point Treatment Plant will receive additional sewage flows that previously were discharged to receiving waterbodies. The effort to convey and treat these additional flows is expected to increase energy consumption at pump stations and the treatment plant by less than 1 percent.

The project energy requirements represent a small portion of the overall regional demand.

10.21.10 Cultural Resources

The project study area is located in the Ballard, Fremont, Wallingford, and Queen Anne neighborhoods of Seattle, and includes approximately 85 historic-age properties. Only two of these are considered eligible for listing in the NRHP. Additionally, there are three historical districts adjacent to, or overlapping portions of the study area. The identified historical properties eligible for listing are not located within these districts.

Project plans will directly impact two unevaluated historic properties. The potentially eligible properties are located adjacent to the West Portal and Ballard conveyance: the Ballard Terminal Railroad alignment and the Stimson Lumber Company Office building. Improvements to the Ballard Terminal Railroad to allow for transportation of project spoils are not expected to cause a significant probable impact. Typically, an NRHP-eligible railroad is not considered diminished if expanded. Construction in the right-of-way in front of the Stimson Lumber Company Office will likely involve increased dust or vibration, but this is not anticipated to be a significant impact. Assessment is recommended for both direct and indirect impacts to historic-aged properties.

In order to comply with Seattle Municipal Code 25.05.675.H, the City-owned public 24th Avenue NW pier, which was built in 1935 and will be directly impacted by the Ship Canal WQ Project, will need to be documented before it can be reconstructed.

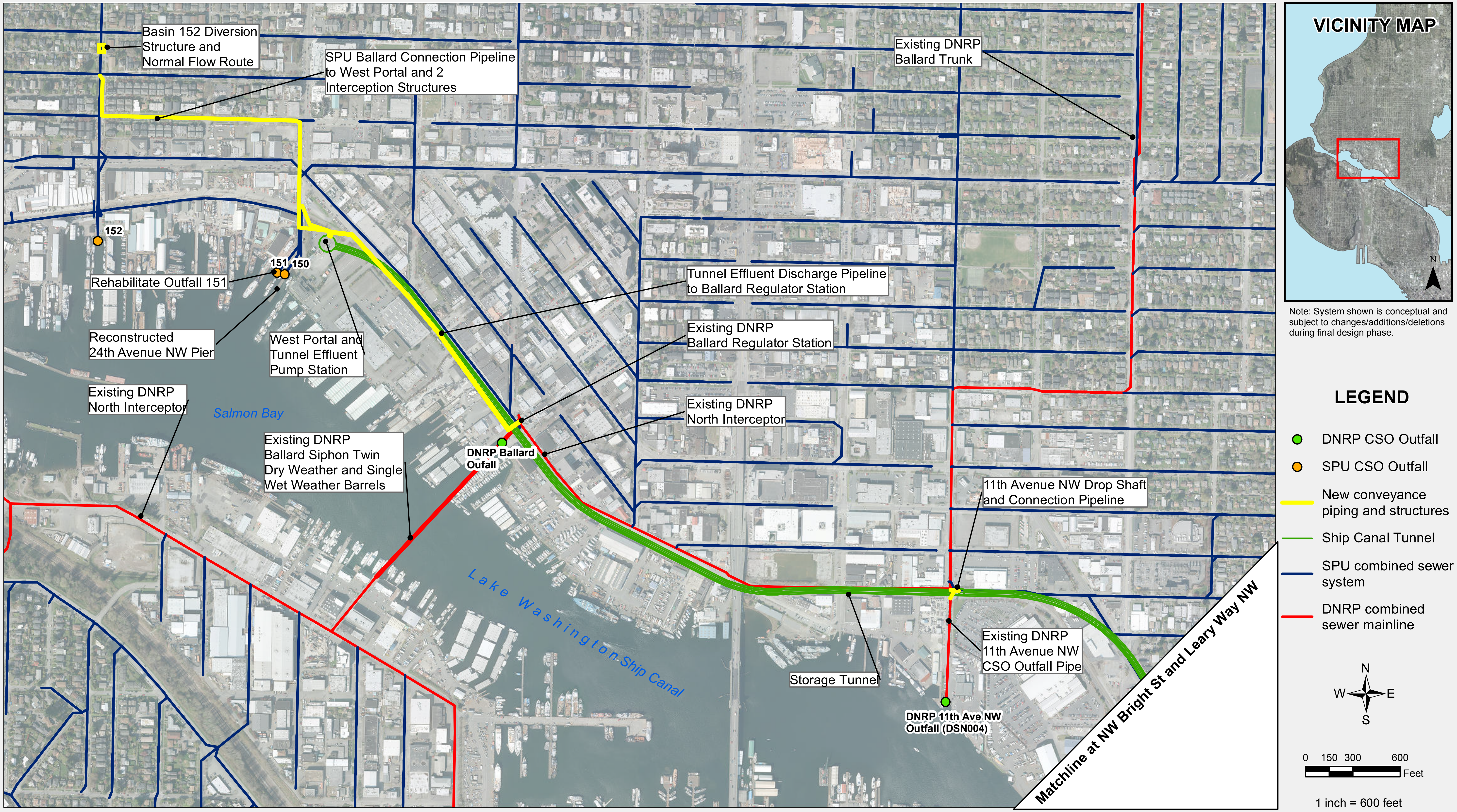
Two study areas were considered: an aboveground cultural resources study area and a study area for archeological resources. The study area for aboveground cultural resources includes the locations of the TEPS at the West Portal, above-grade diesel-powered generator housing at the East Portal, and associated open-cut excavations at each end of the Storage Tunnel. The study area for archaeological cultural resources is the footprint of the tunnel portals, conveyance, and other near-surface impacts plus each adjacent parcel. The study area also includes the conceptual locations of the drop shafts. The storage tunnel alignment is not included in the archaeological or aboveground study areas, because the proposed tunnel depth is within Pleistocene soils and, therefore, predates human occupation of the Puget Sound region.

No archaeological sites are recorded within the study area; therefore, no construction impacts on archaeological resources are anticipated. Although no subsurface survey has been conducted in the study area, several DNRP wastewater facilities, including the Ballard Siphon, have been archaeologically monitored. No cultural resources were identified during monitoring activities (Lockwood and Hoyt, 2012). WISAARD includes a statewide predictive model for precontact archaeology; the archaeological study area is considered “high risk” and “very high risk” for buried cultural resources. Buried cultural resources could include precontact sites, such as Native American encampments, resource procurement sites, food processing sites, or historic buried resources, including foundations and historic abandoned infrastructure, privies, and dumps. These might be present as deep as 25 feet below the present-day ground surface. A review of geological maps suggests that the tunnel itself would not intersect cultural deposits because it will be constructed within pre-Holocene soils.

If archaeological resources were identified during construction, potential impacts to archaeological resources would be permanent because the resources are assumed to be displaced from their context during construction. Near-surface ground disturbance that affects Holocene-aged sediments and historical fill deposits has the potential to affect archaeological resources.

No archaeological sites have been recorded within the study area; however, no surveys have been conducted. Archaeological monitoring is recommended for excavation in intact Holocene strata.

Operational impacts to historic resources might include permanent visual impacts or operational odor, noise, or vibration. Based on preliminary design information, no significant probable operational impacts are expected to aboveground historic resources. No operational impacts to archaeological resources are expected.

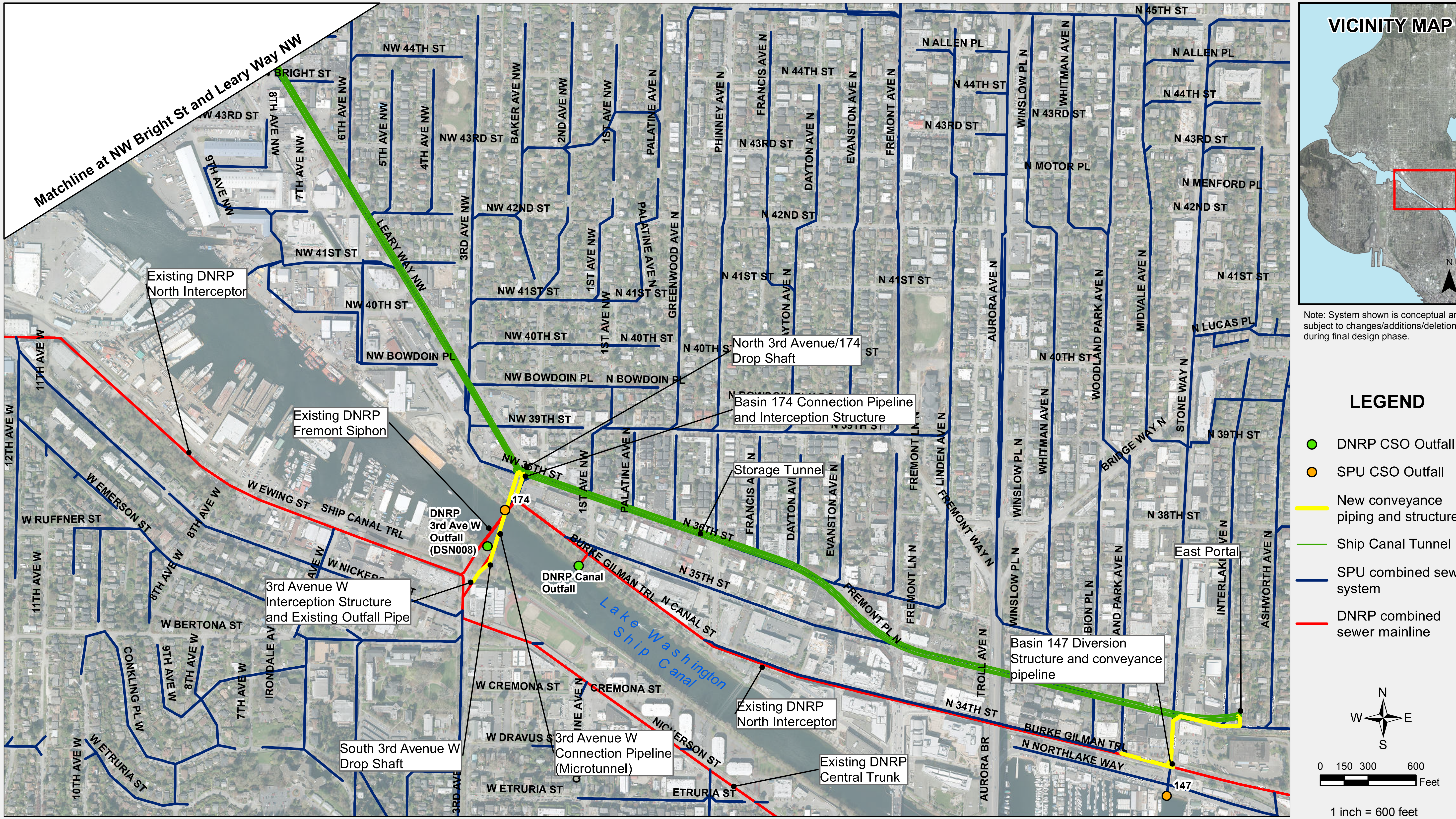


Seattle Public Utilities

Ship Canal Water Quality Project
Final Facility Plan

RECOMMENDED OPTION OVERVIEW WEST

FEBRUARY 2018	**FIGURE 10-1**



Seattle Public Utilities

Ship Canal Water Quality Project
Final Facility Plan

RECOMMENDED OPTION OVERVIEW EAST

FEBRUARY 2018

FIGURE 10-2

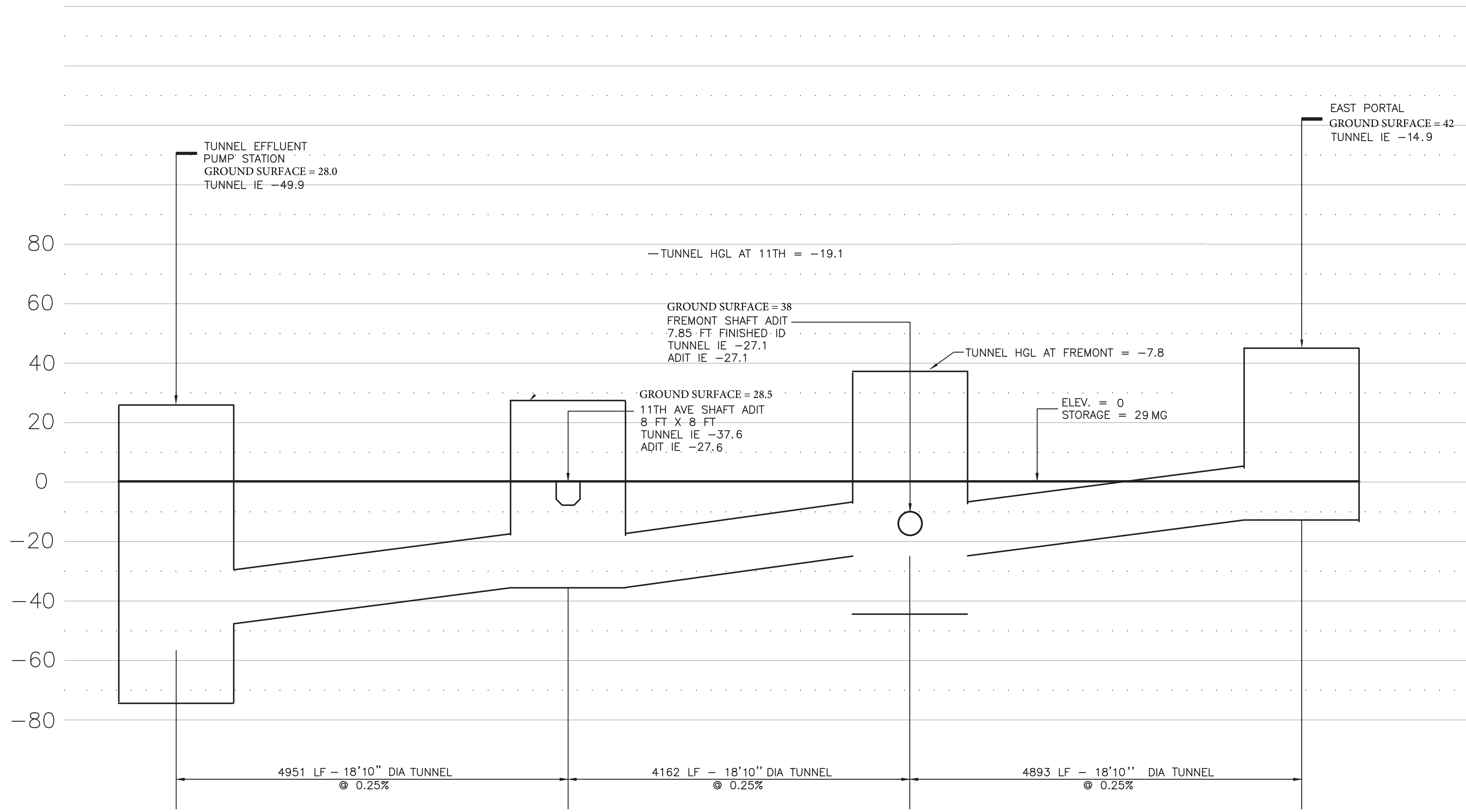


FIGURE 10–3
HYDRAULIC PROFILE
STORAGE TUNNEL

FACILITY PLAN - NOT FOR CONSTRUCTION

ch2m: UrbanTech SYSTEMS

NAVD88
NAD83/91

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REVIEWED BY SPU/DRAINAGE .... 20.....
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King County

SHIP CANAL WATER QUALITY PROJECT

JOB NO. PC C314056
R/W
CO C314056
VAULT PLAN NO.
SHEET OF

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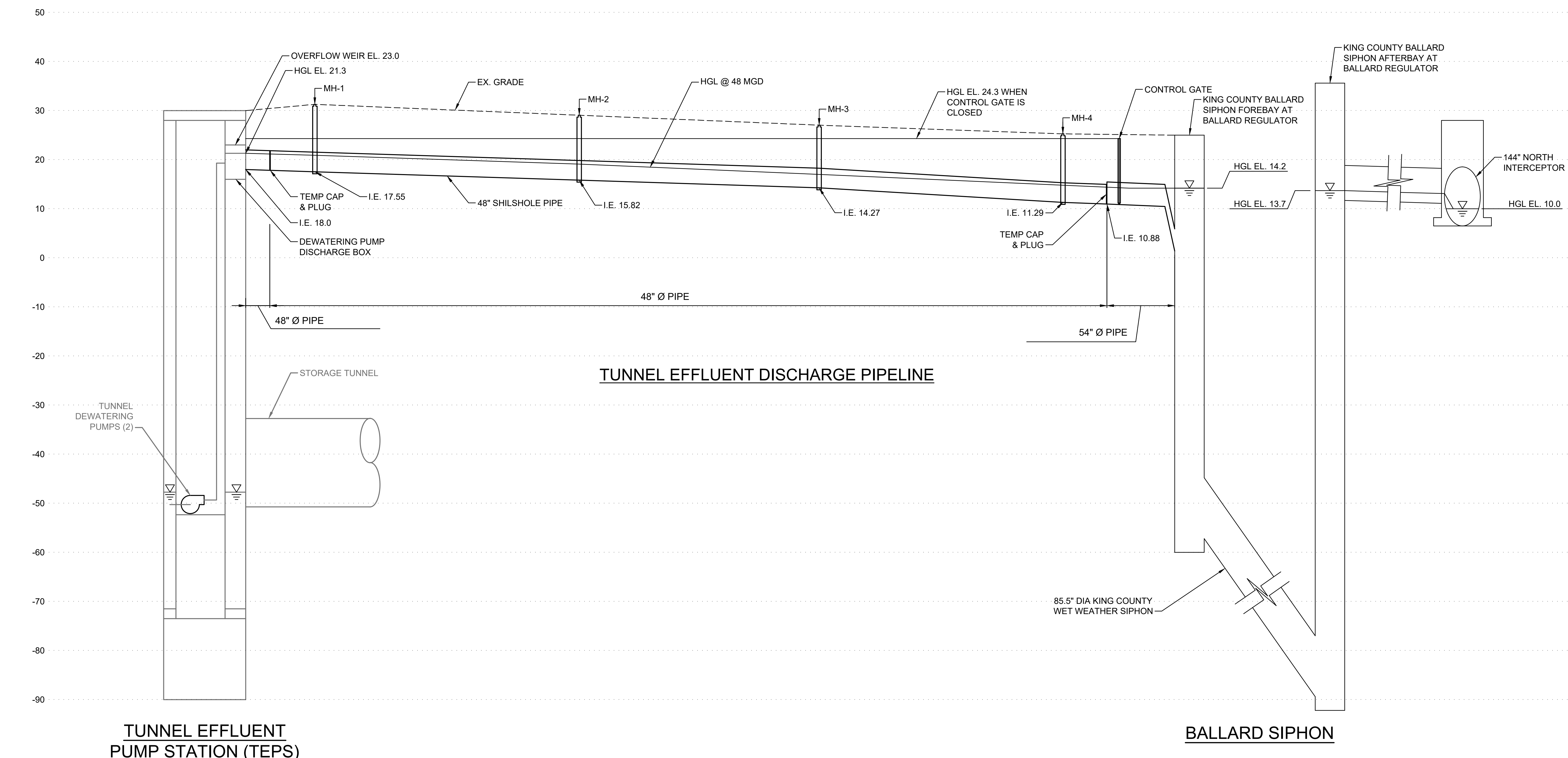

H:NAD 83/91
V:NAVD-88

90% REVIEW SUBMITTAL (NOT FOR CONSTRUCTION)

TUNNEL EFFLUENT DISCHARGE PIPELINE HYDRAULIC PROFILE 1

UrbanTech SYSTEMS Consulting Engineering & Construction Management

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C O S ROBERTS STATE OF WASHINGTON #36518 REGISTERED PROFESSIONAL ENGINEER

Seattle Department of Transportation
ORDINANCE NO. PW NO. ####
SCALE: H. 1"=20', V. 1"=10'

BURKE-GILMAN TRAIL MISSING LINK

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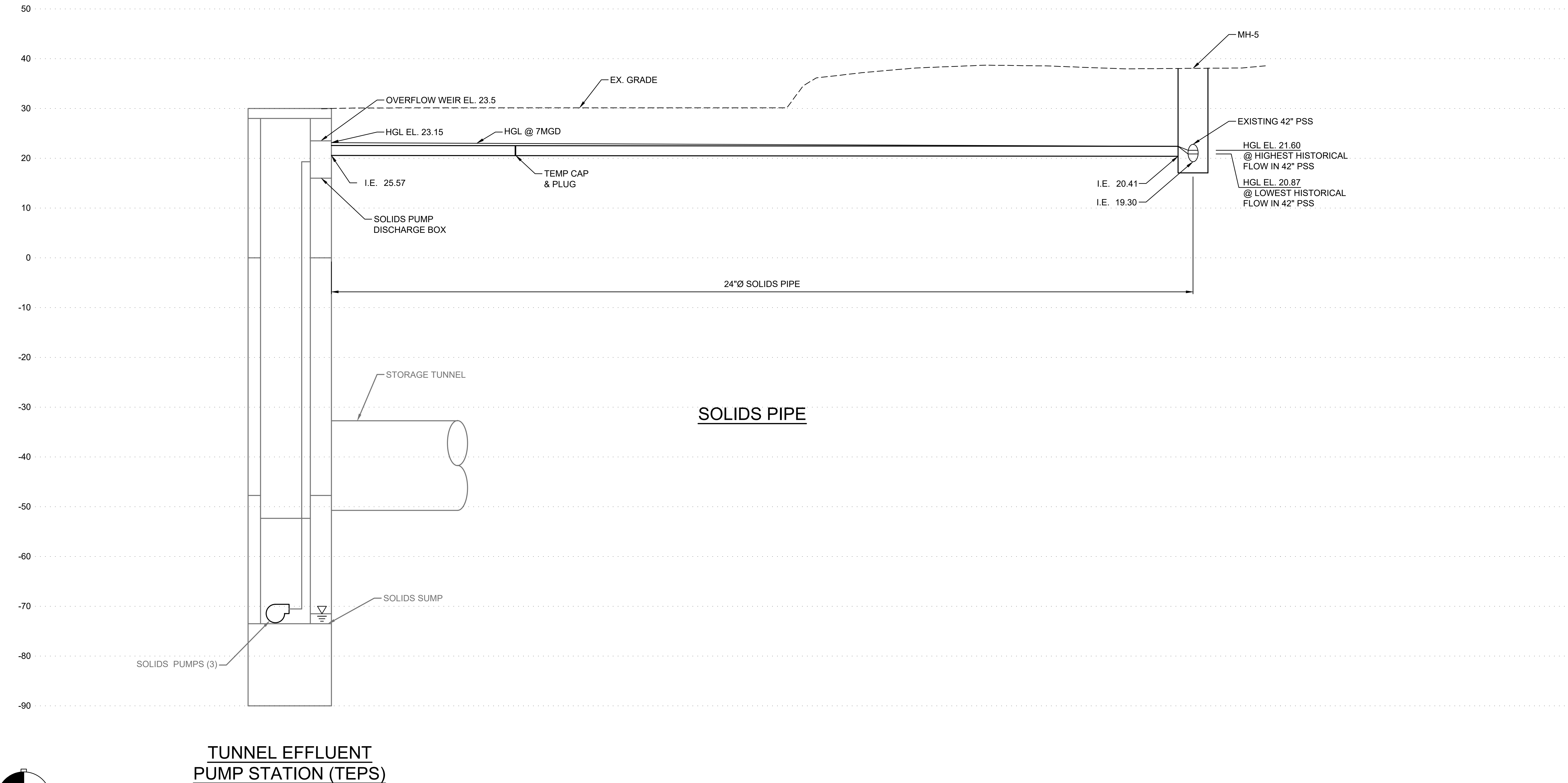

90% REVIEW SUBMITTAL (NOT FOR CONSTRUCTION)

SHILSHOLE PIPE - HYDRAULIC PROFILE 2

UrbanTech SYSTEMS Consulting Engineering & Construction Management

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BURKE-GILMAN TRAIL
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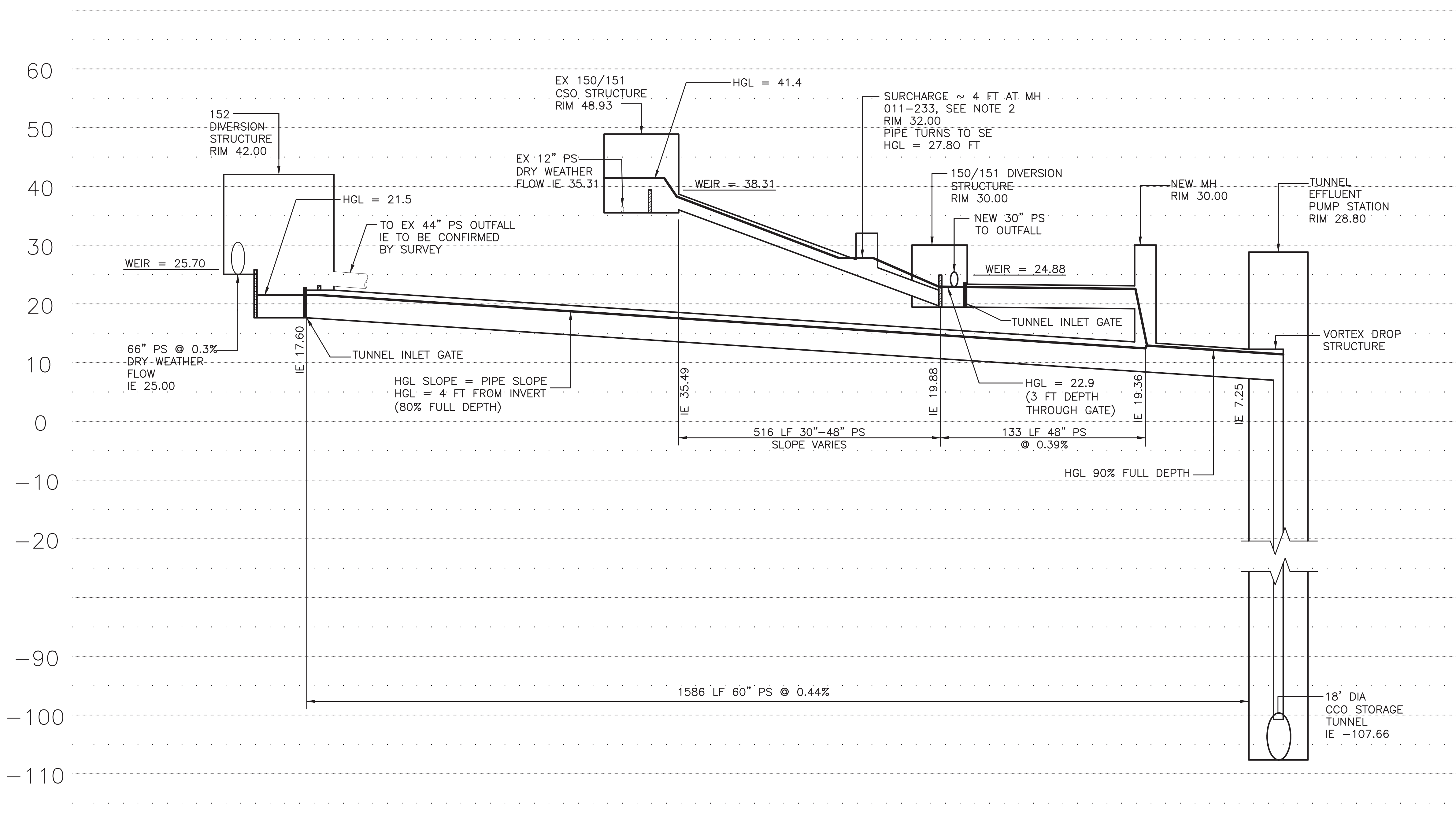

NOTES

1. HGL SHOWN IS THE MAXIMUM VALUE FOR THE 1990 TO 2009 SWMM MODEL SIMULATION DURING PERIODS OF ACTIVE FLOW DIVERSION TO THE TUNNEL. FOR DRY WEATHER CONDITIONS, THE DIVERSION PIPING CARRIES NO FLOW.
2. SPU COULD ELIMINATE SURCHARGE BY ENLARGING PIPE FROM 011−233 TO THE DIVERSION STRUCTURE OR BY MODIFYING THE ALIGNMENT. THE CONVEYANCE PIPE TO 011−233 IS STEEPLY SLOPED ALONG 24TH AVE NW AND THEN TRANSITIONS TO <0.5%. PLEASE NOTE, THERE ARE NO SIDE SEWER CONNECTIONS ALONG THE SURCHARGED PIPE.

FIGURE 10−5
HYDRAULIC PROFILE
OUTFALLS 150/151/152 CONNECTION PIPELINES

FACILITY PLAN - NOT FOR CONSTRUCTION

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ch2m

NAVD88
NAD83/91

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King County

SHIP CANAL WATER QUALITY PROJECT

JOB NO. PC C314056
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VAULT PLAN NO.
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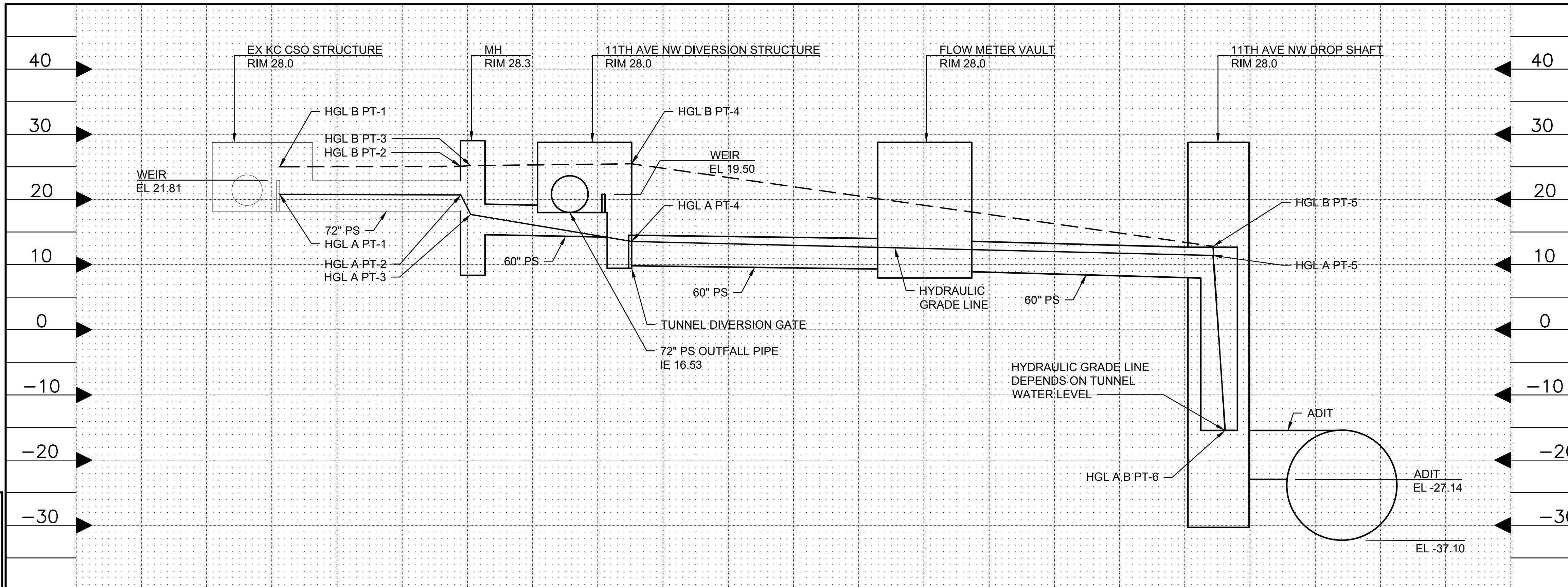

NOTES:

1. HGL IS BASED ON THE NOV 3, 1978 STORM EVENT.
2. HGL CONDITION A IS THE HGL AT THE MAXIMUM FLOW THROUGH THE DIVERSION GATE, JUST BEFORE THE GATE CLOSES. THIS IS THE HGL STORM SHOWN ON THE PROFILE.
3. HGL CONDITION B IS THE HGL AT THE MAXIMUM FLOW, JUST AFTER THE DIVERSION GATE HAS CLOSED, THE TUNNEL IS FULL, AND THERE IS OVERLOW TO THE OUTFALL. DOWNSTREAM OF THE DIVERSION GATE, THE HGL IS THE PIPE INVERT OR THE DEPTH OF STORED WATER, WHICHEVER IS HIGHER.

	HGL CONDITION A	HGL CONDITION B
HGL PT-1	19.5	24.0
HGL PT-2	19.4	24.1
HGL PT-3	16.2	24.2
HGL PT-4	11.8	24.5
HGL PT-5	9.5	11.0
HGL PT-6	-19.1	-19.1

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V: NAVD-88

FACILITY PLAN (NOT FOR CONSTRUCTION)

GEN-G-311
GENERAL
11TH AVE SHAFT SITE HYDRAULIC PROFILE

Brown AND Caldwell
McMILLEN JACOBS ASSOCIATES

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ROBERT WILLIAM JACKSON
STATE OF WASHINGTON
46074
REGISTERED
PROFESSIONAL ENGINEER

Seattle Public Utilities
ORDINANCE NO. PW NO. 2018-018
FUND:
SCALE: NONE

SCWQP: STORAGE TUNNEL

JOB NO. PC
R/W
CO C315503
VAULT PLAN NO.
777-905
SHEET OF

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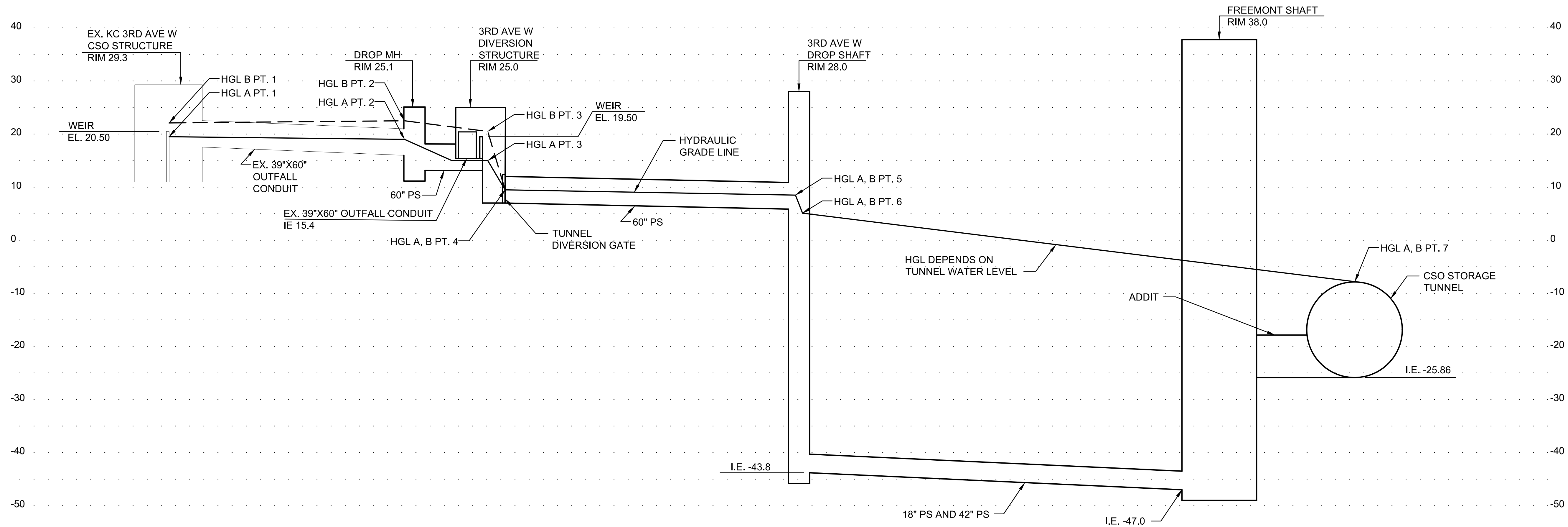

	HGL CONDITION A	HGL CONDITION B
HGL PT. 1	19.5	22.1
HGL PT. 2	19.0	22.5
HGL PT. 3	15.0	20.5
HGL PT. 4	9.5	9.5
HGL PT. 5	8.5	8.5
HGL PT. 6	5.1	5.1
HGL PT. 7	-7.8	-7.8

NOTES:

1. HGL IS BASED ON THE NOV 3, 1978 STORM EVENT.
2. HGL CONDITION A IS THE HGL AT THE MAXIMUM FLOW THROUGH THE DIVERSION GATE, JUST BEFORE THE GATE CLOSES. THIS IS THE HGL STORM SHOWN ON THE PROFILE.
3. HGL CONDITION B IS THE HGL AT THE MAXIMUM FLOW, JUST AFTER THE DIVERSION GATE HAS CLOSED, THE TUNNEL IS FULL, AND THERE IS OVERLOW TO THE OUTFALL. DOWNSTREAM OF THE DIVERSION GATE, THE HGL IS THE PIPE INVERT OR THE DEPTH OF STORED WATER, WHICHEVER IS HIGHER.

H: NAD 83-2011
V: NAVD-88

GEN-G-316
GENERAL
3RD AVE SHAFT SITE HYDRAULIC PROFILE

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McMILLEN JACOBS ASSOCIATES

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CITY PURCHASING & CONTRACTING SERVICES DIRECTOR

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CHECKED REB
DRAWN NSW
CHECKED BLB

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REVIEWED:
DES. CONST.
SDOT PROJ. MGR.
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REVISED AS BUILT

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ROBERT WILLIAM JACKSON STATE OF WASHINGTON 46074 REGISTERED PROFESSIONAL ENGINEER

Seattle Public Utilities
ORDINANCE NO. PW NO. 2018-018
FUND:
SCALE: NONE

SCWQP: STORAGE TUNNEL

JOB NO. PC
R/W
CO C315503
VAULT PLAN NO. 777-905
SHEET OF

MARK | DATE | NATURE REVISIONS | MADE | CHKD | REV'D

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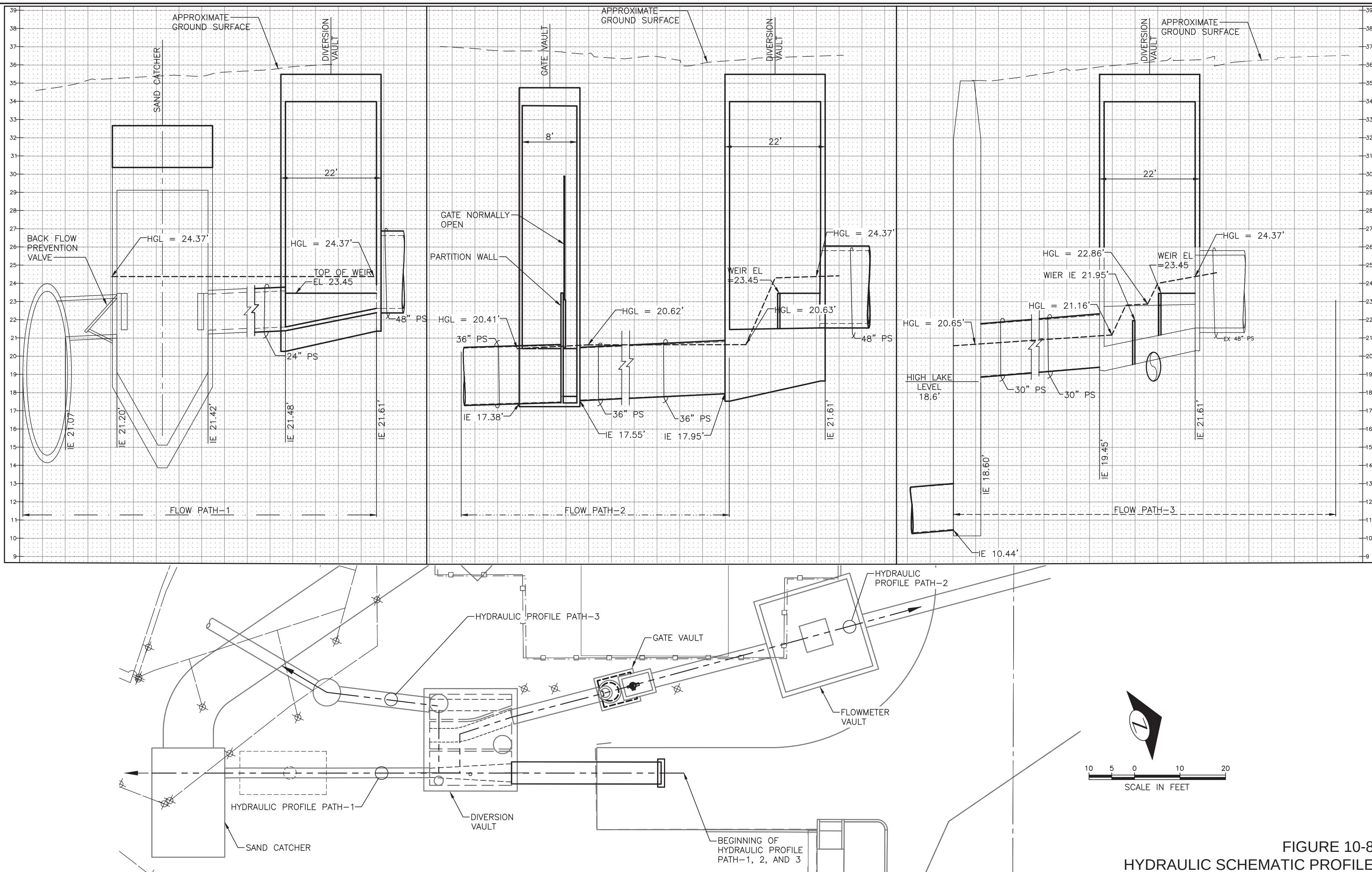

FIGURE 10-8
HYDRAULIC SCHEMATIC PROFILE

FACILITY PLAN - NOT FOR CONSTRUCTION

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BY: . . . .
CITY PURCHASING & CONTRACTING SERVICES DIRECTOR

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DESIGNED PJM CHECKED	REVIEWED: DES. CONST. SDOT PROJ. MGR.
DRAWN JCJ CHECKED TMR	RECEIVED REVISED AS BUILT

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PATRICK J. MURPHY — STATE OF WASHINGTON — 27456 — REGISTERED PROFESSIONAL ENGINEER

SCWQP: FREMONT CONVEYANCE

JOB NO. PC C315504
R/W
CO
VAULT PLAN NO. 777-905
SHEET 6 OF 23

DATE	MARK	NATURE	MADE	CHK'D	REV'D
		REVISIONS			

VAULT SERIAL NO. 36947

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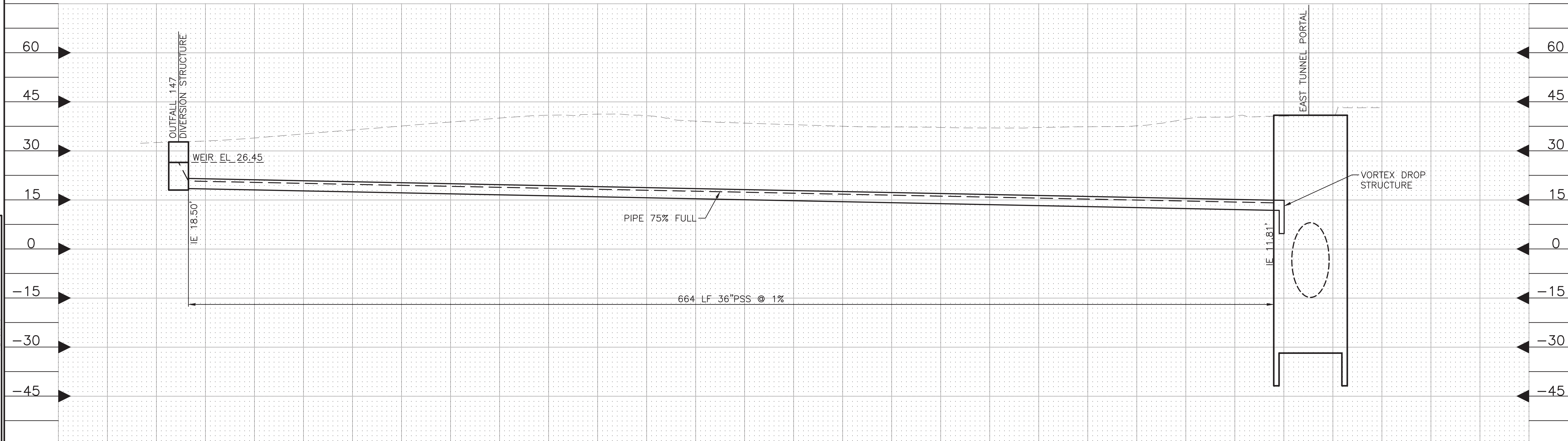

SCALE H: 1"=30' V:1"=15'

FIGURE 10–9
HYDRAULIC PROFILE
OUTFALL 147 CONNECTION PIPELINE

DATE	MARK	NATURE / REVISIONS	MADE	CHK'D	REV'D

VAULT SERIAL NO.

SDOT UMP NO. 352063

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DESIGNED CHECKED	REVIEWED: DES. CONST. SDOT PROJ. MGR.
DRAWN CHECKED	RECEIVED REVISED AS BUILT

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Seattle Public Utilities

ORDINANCE NO. PW NO.
FUND:
SCALE: AS NOTED

SHIP CANAL WATER QUALITY PROJECT

JOB NO.	
PC	C314056
R/W	
CO	C314056

VAULT PLAN NO.

SHEET OF

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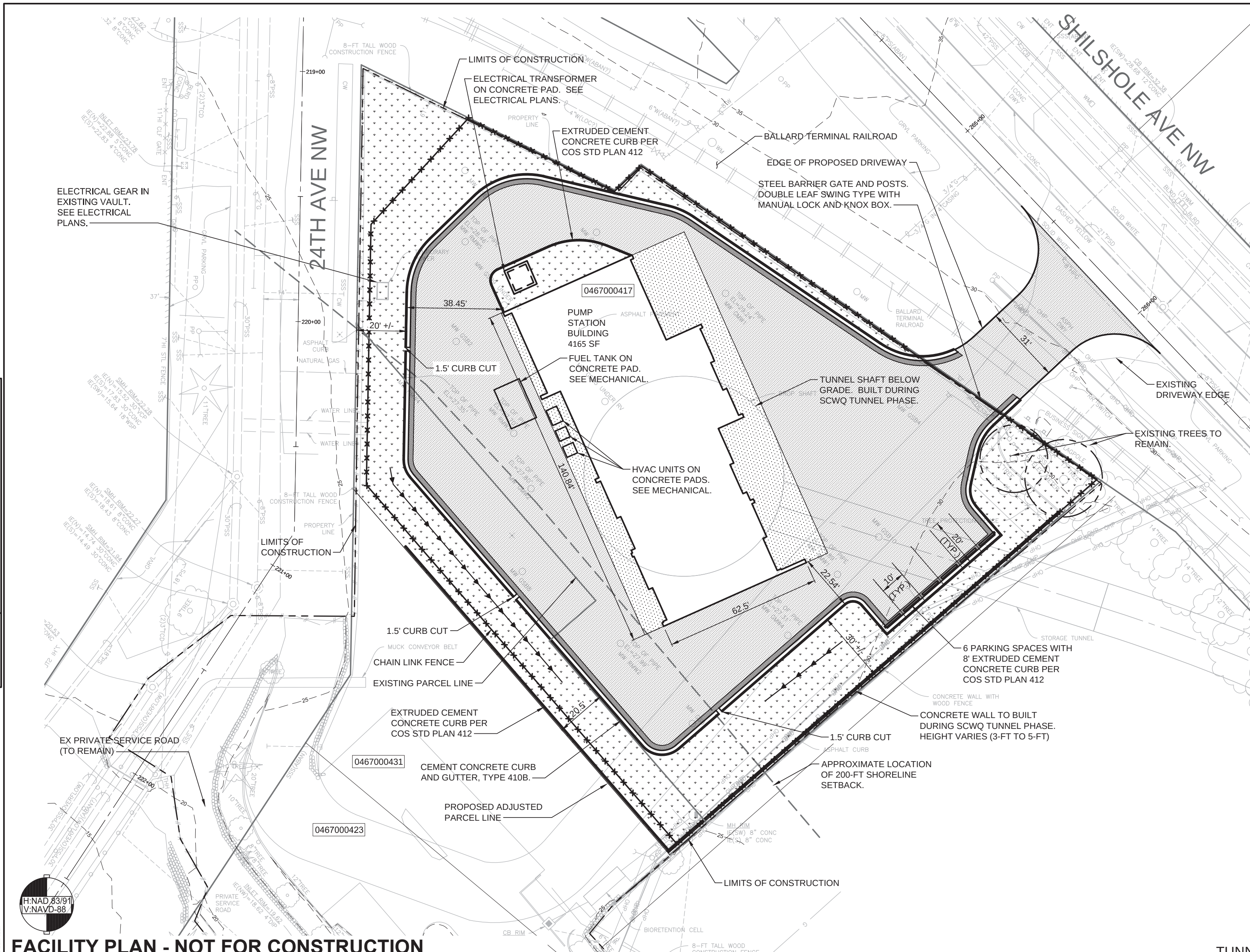

N

20 10 0 20 40

SCALE IN FEET

NOTES

UNLESS OTHERWISE NOTED

1. CONTRACTOR TO FIELD VERIFY AND LOCATE ALL UTILITIES PRIOR TO BEGINNING WORK AND NOTIFY ENGINEER OF ANY DISCREPANCIES. CONTACT UTILITIES UNDERGROUND LOCATION CENTER AT 1-800-424-555 OR 811.
2. WEST SHAFT AND TUNNEL CONSTRUCTION TO OCCUR PRIOR TO PUMP STATION CONSTRUCTION. LOCATION OF SHAFT AND EXISTING SITE FEATURES TO BE VERIFIED AFTER COMPLETION OF WEST SHAFT CONSTRUCTION AND PRIOR TO COMMENCEMENT OF PUMP STATION CONSTRUCTION.
3. APPROXIMATE SITE ADDRESS: 5425 SHILSHOLE AVE NW.
4. KING COUNTY PARCEL # 0467000417 AND #0467000431.
5. FIRE ACCESS FOR A 100-FT AERIAL 500# TIP TILLER TRUCK EMERGENCY VEHICLE HAS BEEN VERIFIED USING AUTOTURN.

LEGAL DESCRIPTION:

1. PARCEL #0467000417 (NORTHERNMOST PARCEL): BALLARD TIDE LANDS PCL Y SEATTLE BLA #3011659 REC #20140108900001 SD BLA BEING POR BLOCK 8 OF SD ADD TGW UNPL PROP ADJ
2. PARCEL #0467000431 (MIDDLE PARCEL): BALLARD TIDE LANDS PCL Z SEATTLE BLA #3011659 REC #20140108900001 SD BLA BEING POR BLOCK 8 OF SD ADD TGW UNPL PROP ADJ
3. PARCEL #0467000423 (YANKEE DINER): BALLARD TIDE LANDS THAT POR LOTS 1-2-3 BLK 8 DAF BEG MOST WLY COR SD BLK 8 TH N 32-22-11 E ALG NWLY LN THOF 135.67 FT TO THE TPOB TH S 42-21-43 E 149.69 FT TH N 48-05-37 E 148.07 FT TH N 41-54-23 W 18.44 FT TH N 48-05-37 E 50 FT M/L TO GOVT MDR LN TH NWLY ALG SD GOVT MDR LN TO SE LN OF 24TH AV NW TH SWLY ALG SD SE LN TO THE POB

NOTES TO REVIEWER:

1. LIMITS OF STREET RESTORATION WITHIN RIGHT OF WAY WILL BE INCLUDED IN CONSTRUCTION DOCUMENT SET AT 60% SUBMITTAL.

SITE LEGEND:

##	PROPERTY NUMBER
	LIMITS OF CONSTRUCTION
	CHAIN LINK FENCE
	PROPOSED FULL DEPTH ASPHALT PAVEMENT
	PROPOSED 3" ASPHALT OVERLAY
	6" CONCRETE PAVEMENT
	LANDSCAPE AREA SEE LANDSCAPE PLANS

FACILITY PLAN - NOT FOR CONSTRUCTION

FIGURE 10-10

TUNNEL EFFLUENT PUMP STATION - FINISHED SITE PLAN

VAULT SERIAL NO.	DATE	MARK	NATURE	MADE	CHKD	REVD
			REVISIONS			

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Pete Jacobchuk Jun-03-16 10:43am

EES ENTITLEMENT & ENGINEERING SOLUTIONS, Inc.

McMILLEN JACOBS ASSOCIATES

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NANCY LOCKE
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SEATTLE, WASHINGTON 20
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CITY PURCHASING & CONTRACTING SERVICES DIRECTOR

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DESIGNED AMP		REVIEWED:	
CHECKED		DES.	CONST.
		SDOT	PROJ. MGR.
DRAWN JAG		RECEIVED	
CHECKED AMH		REVISED AS BUILT	

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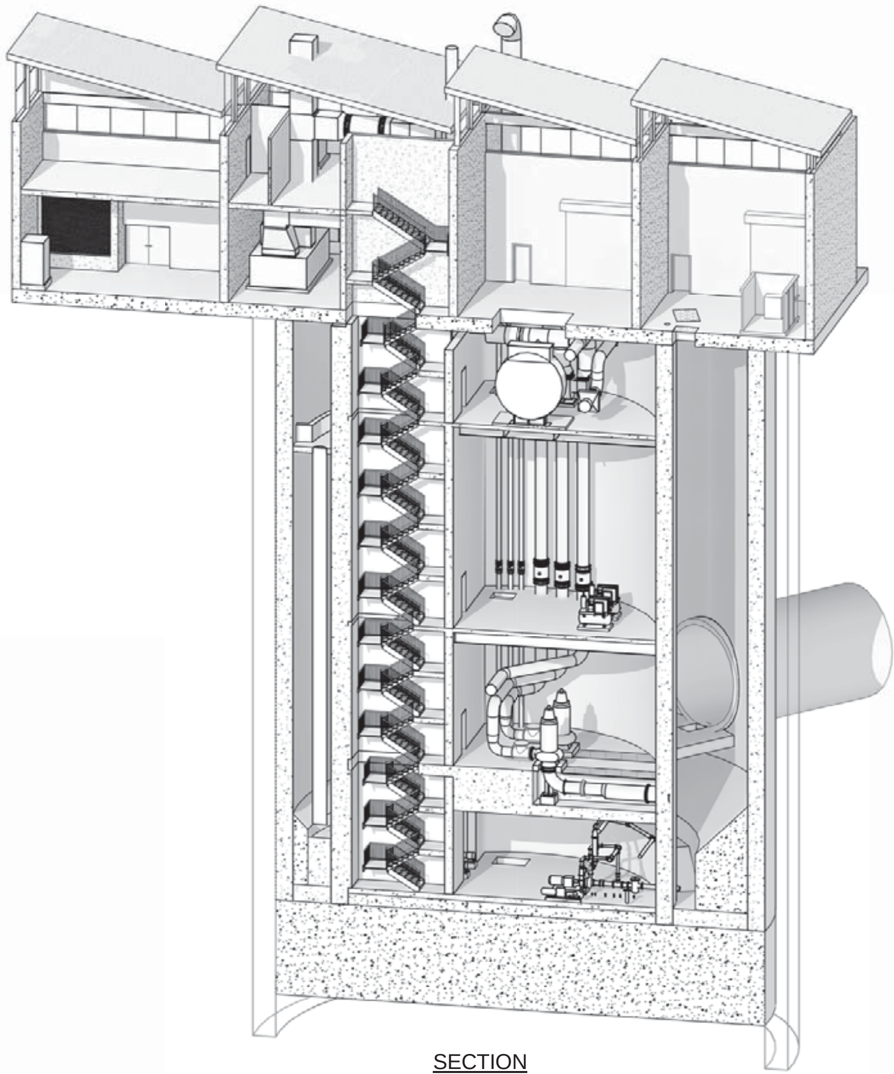
ENGINEER'S NAME
STATE OF WASHINGTON
REGISTERED PROFESSIONAL ENGINEER

SHIP CANAL WATER QUALITY PROJECT

JOB NO. PC R/W CO

VAULT PLAN NO.

SHEET C-130 OF 70



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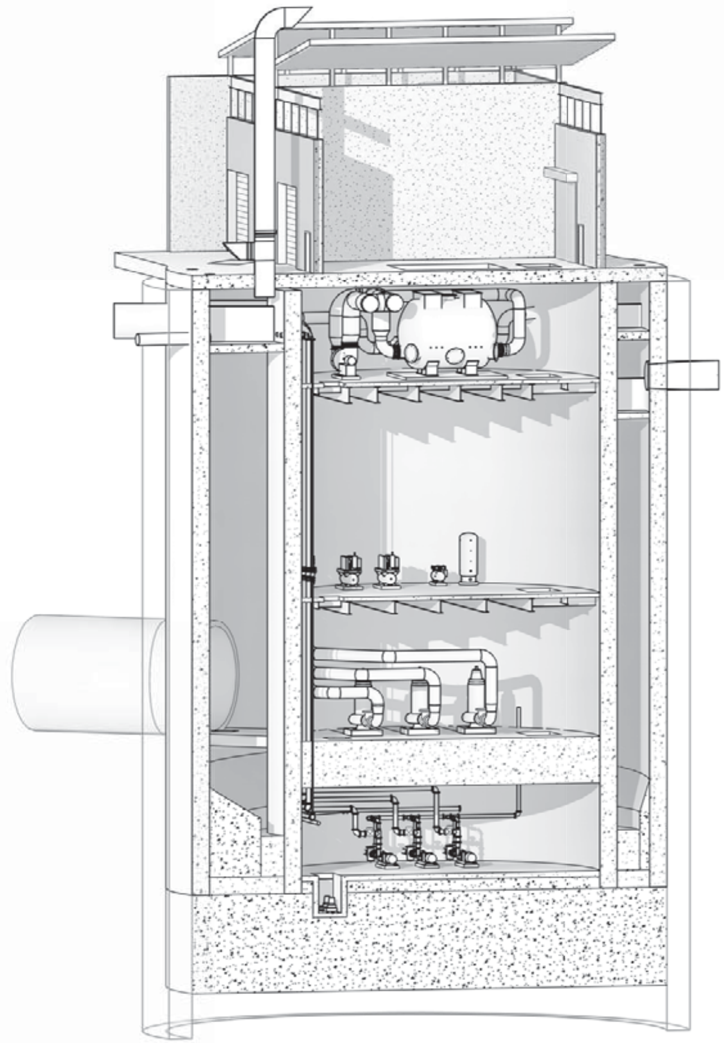

SECTION
SCALE: NONE

H:NAD 83/91
V:NAVD-88

VAULT SERIAL NO.	DATE	MARK	NATURE	MADE	CHK'D	REV'D
			REVISIONS			

FACILITY PLAN - NOT FOR CONSTRUCTION

FIGURE 10-11
TUNNEL EFFLUENT PUMP STATION - 3D VIEWS

HDR
McMILLEN JACOBS ASSOCIATES

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PURCHASING & CONTRACTING SERVICES DIRECTOR

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DRAWN Author CHECKED Checker	RECEIVED REVISED AS BUILT

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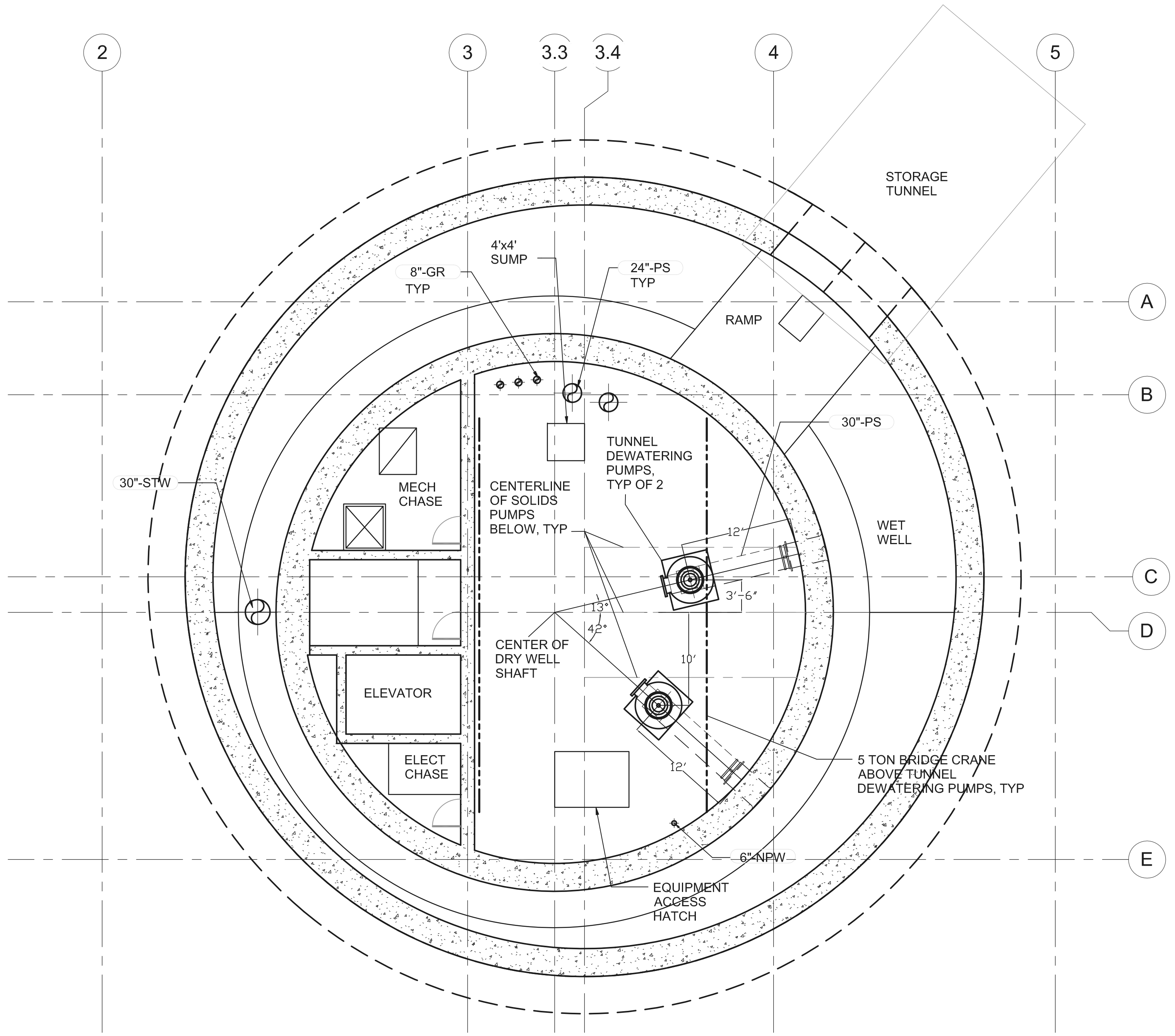
SHIP CANAL WATER QUALITY PROJECT

JOB NO.
PC ---
R/W ---
CO ---

VAULT PLAN NO.

SHEET 62 OF 70



NOTES:

1. TUNNEL DEWATERING PUMP DISCHARGE PIPING NOT SHOWN.

SCALE: 1/8"=1'-0"

DEWATERING PUMP LEVEL - EL -50.50

SCALE: 1/8" = 1'-0"

H:NAD 83/91
V:NAVD-88

30% REVIEW SUBMITTAL (NOT FOR CONSTRUCTION)

M-102
MECHANICAL PLAN - DEWATERING PUMP LEVEL

FDR | McMILLEN JACOBS ASSOCIATES

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DRAWN Author
CHECKED Checker

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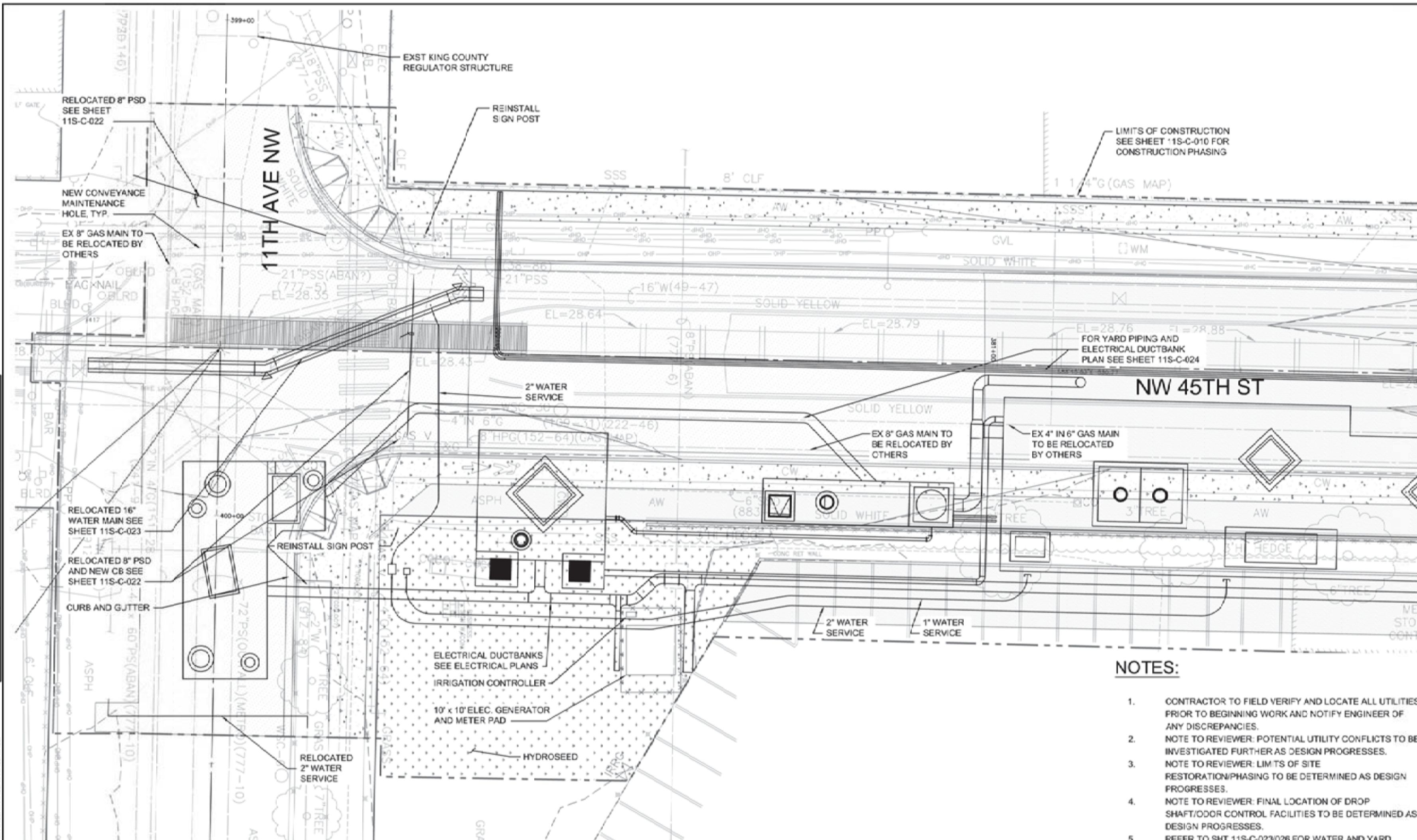
DES CONST
SOOT PROJ. MGR.
RECEIVED:
REVISED AS BUILT

Seattle Public Utilities
City of Seattle
Ray Hoffman, Director
ORDINANCE NO. APPROVED
FUND
SCALE 1/8" = 1'-0" INSPECTOR'S BOOK

SCWQP: TUNNEL EFFLUENT PUMP STATION

PC ---
R/W ---
CO ---
VAULT PLAN NO.

SHEET 52 OF 70



NOTES:

1. CONTRACTOR TO FIELD VERIFY AND LOCATE ALL UTILITIES PRIOR TO BEGINNING WORK AND NOTIFY ENGINEER OF ANY DISCREPANCIES.
2. NOTE TO REVIEWER: POTENTIAL UTILITY CONFLICTS TO BE INVESTIGATED FURTHER AS DESIGN PROGRESSES.
3. NOTE TO REVIEWER: LIMITS OF SITE RESTORATION/PHASING TO BE DETERMINED AS DESIGN PROGRESSES.
4. NOTE TO REVIEWER: FINAL LOCATION OF DROP SHAFT/ODOR CONTROL FACILITIES TO BE DETERMINED AS DESIGN PROGRESSES.
5. REFER TO SHT 11S-C-023/026 FOR WATER AND YARD PIPING
6. FOR SURVEY CONTROL PLANS SEE GEN-G-160 THROUGH GEN-G-184.

FIGURE 10-13
11TH AVE DROP SHAFT - FINISHED SITE PLAN

FACILITY PLAN - NOT FOR CONSTRUCTION

SCE | McMILLEN JACOBS ASSOCIATES

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	SDOT PROJ. MGR.
DRAWN	RECEIVED
CHECKED	REVISED AS BUILT

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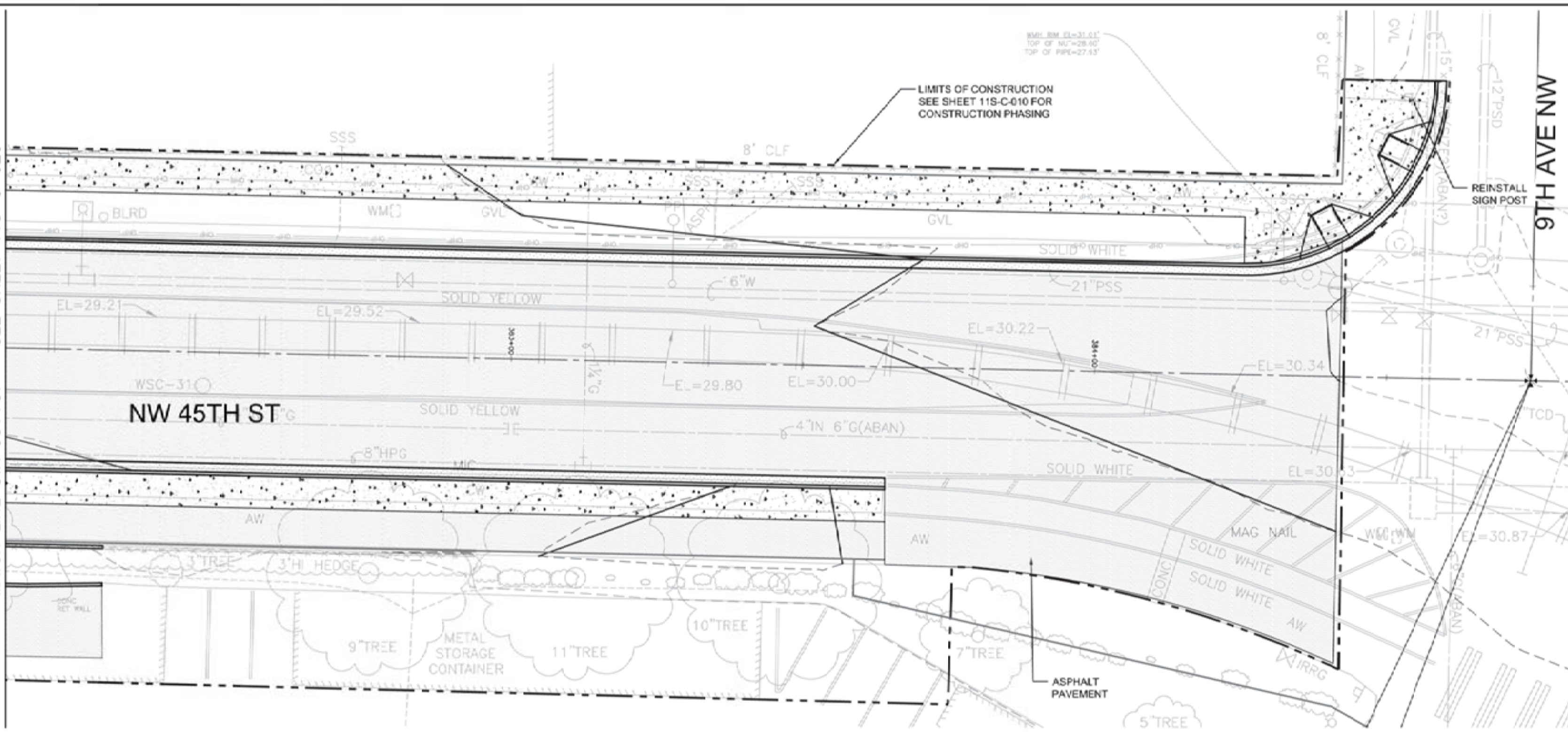
SHIP CANAL WATER QUALITY PROJECT

JOB NO. PC R/W CO
VAULT PLAN NO.
SHEET 164 OF 428

VAULT SERIAL NO.	DATE	MARK	NATURE	MADE	CHK'D	REV'D
			REVISIONS			



NOTES:

1. CONTRACTOR TO FIELD VERIFY AND LOCATE ALL UTILITIES PRIOR TO BEGINNING WORK AND NOTIFY ENGINEER OF ANY DISCREPANCIES.
2. NOTE TO REVIEWER: POTENTIAL UTILITY CONFLICTS TO BE INVESTIGATED FURTHER AS DESIGN PROGRESSES.
3. NOTE TO REVIEWER: LIMITS OF SITE RESTORATION/PHASING TO BE DETERMINED AS DESIGN PROGRESSES.
4. FOR SURVEY CONTROL PLANS SEE GEN-G-160 THROUGH GEN-G-184.

FIGURE 10-14
11TH AVE DROP SHAFT - FINISHED SITE PLAN

FACILITY PLAN - NOT FOR CONSTRUCTION

SCE
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DESIGNED	REVIEWED: DES. CONST
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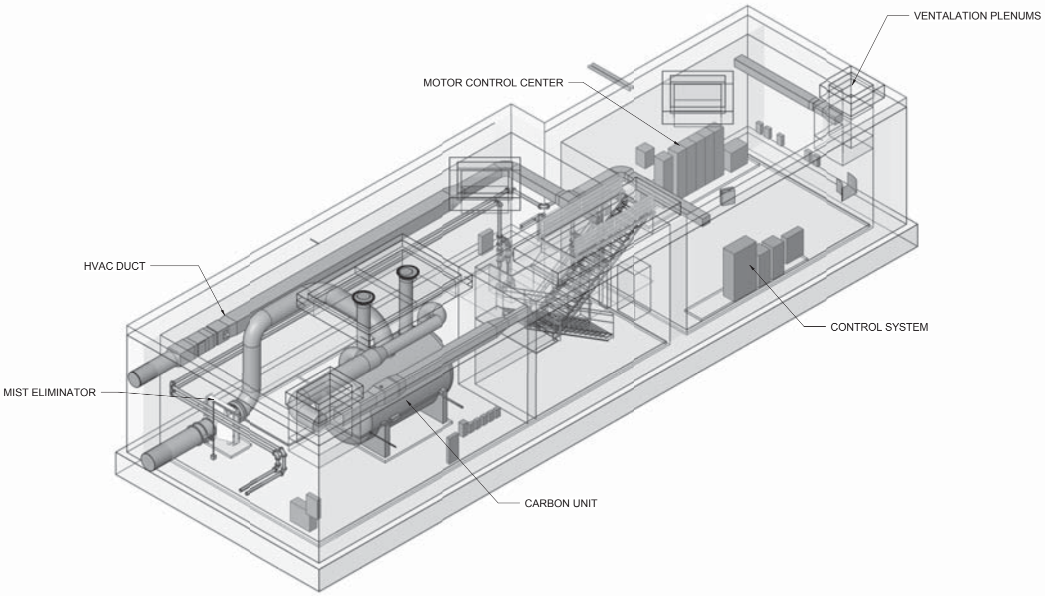
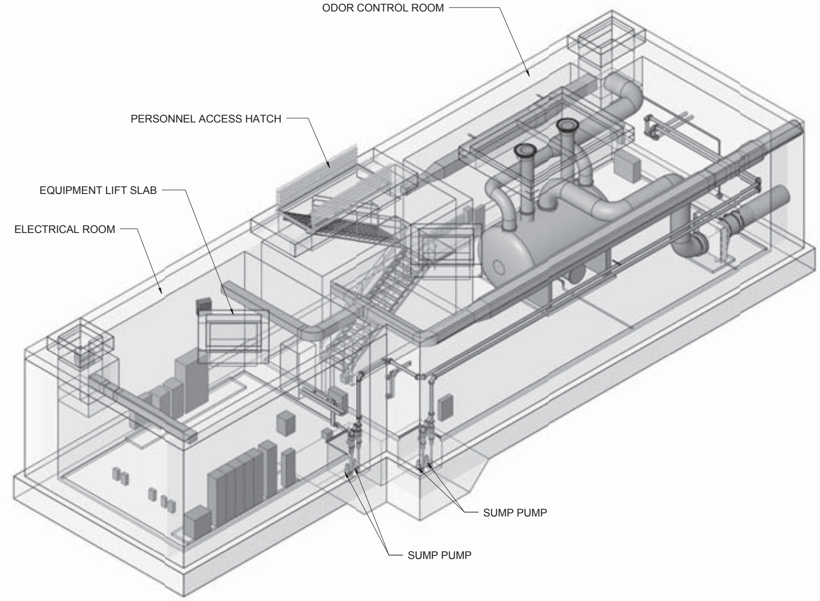
VAULT PLAN NO.
SHEET 165 OF 272

DATE	MARK	NATURE / REVISIONS	MADE	CHK'D	REV'D

VAULT SERIAL NO.



MECHANICAL/ELECTRICAL VAULT - 3D VIEW 1
SCALE: NONE



MECHANICAL/ELECTRICAL VAULT - 3D VIEW 2
SCALE: NONE

H:NAD 83/91
V:NAVD-88

FACILITY PLAN - NOT FOR CONSTRUCTION

FIGURE 10-15
11TH AVE DROP SHAFT - 3D VIEWS

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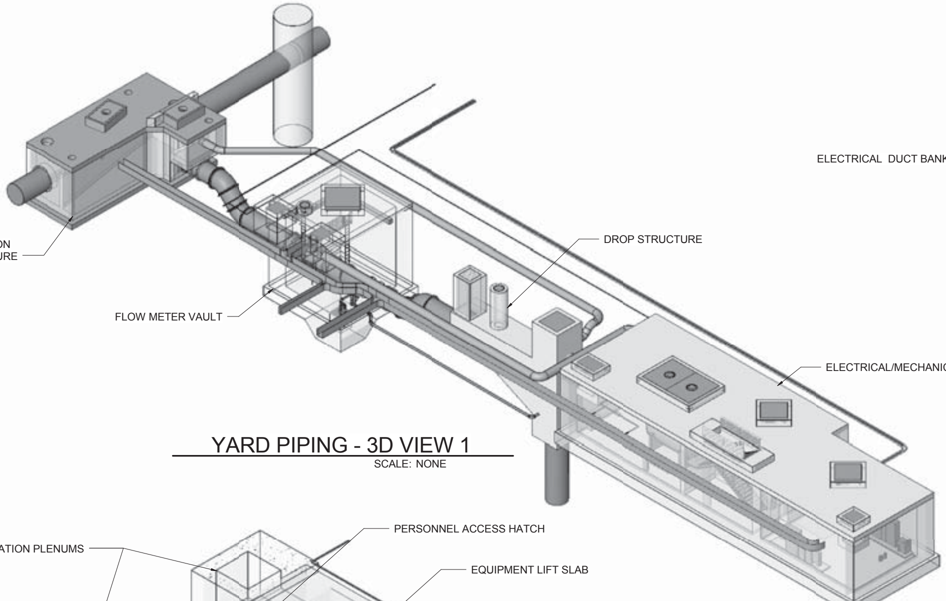
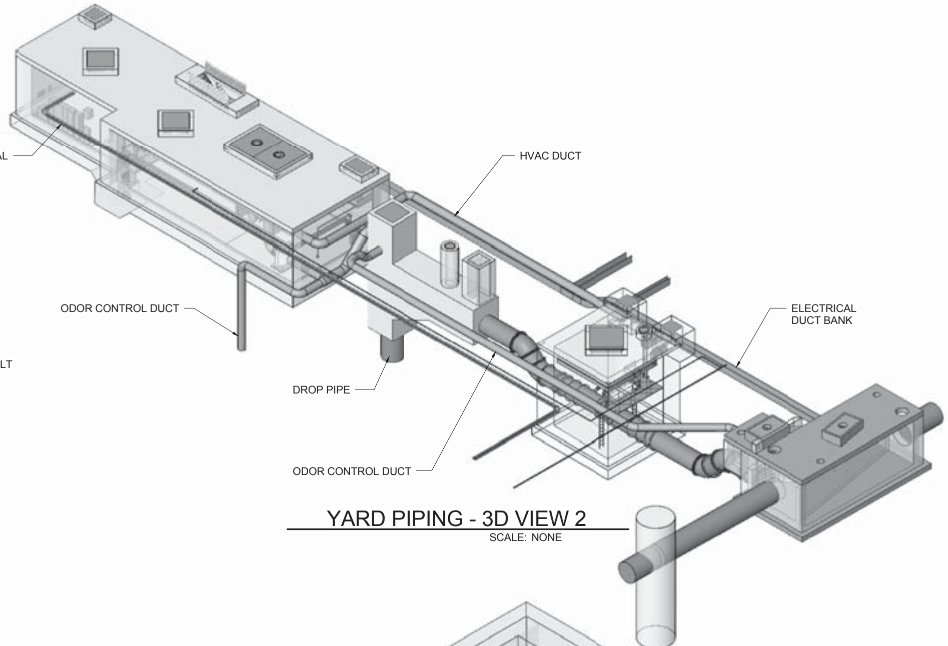
VAULT PLAN NO.
SHEET OF

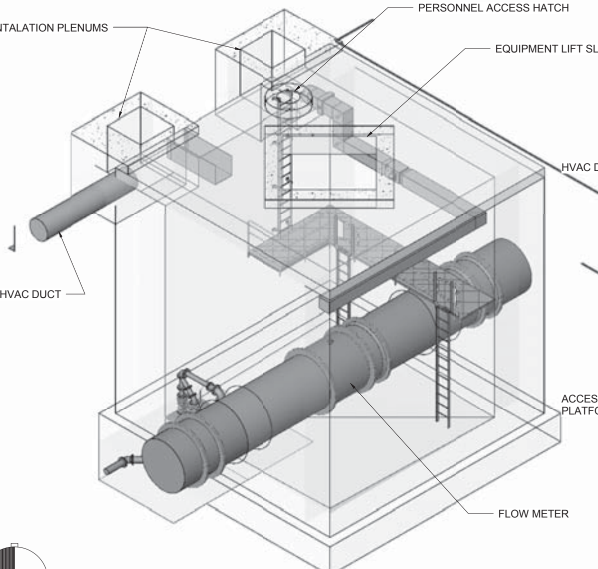
VAULT SERIAL NO. | DATE | MARK | NATURE | MADE | CHK'D | REV'D
REVISIONS



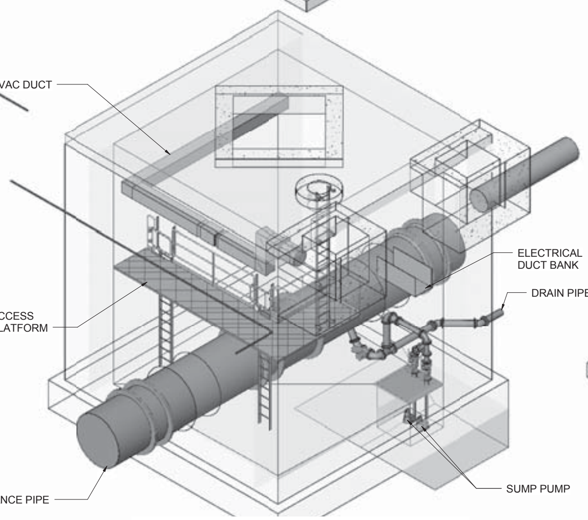
YARD PIPING - 3D VIEW 1
SCALE: NONE



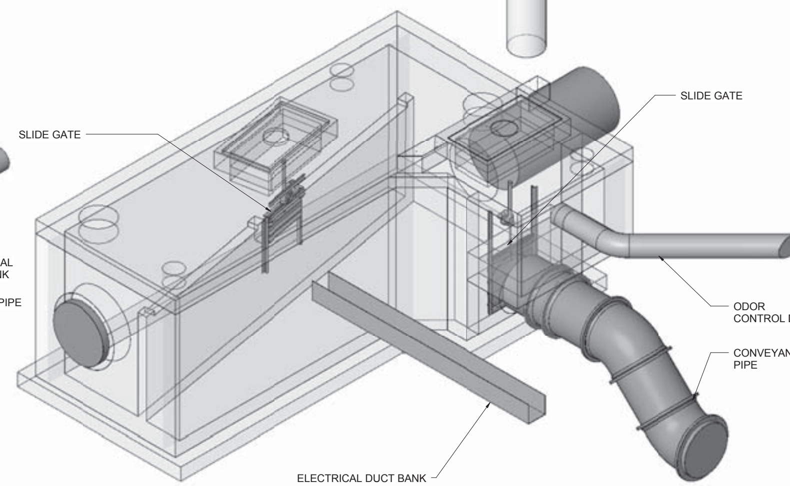
YARD PIPING - 3D VIEW 2
SCALE: NONE



FLOW METER VAULT - 3D VIEW 3
SCALE: NONE



FLOW METER VAULT - 3D VIEW 4
SCALE: NONE



DIVERSION STRUCTURE - 3D VIEW 5
SCALE: NONE

H:NAD 83/91
V:NAVD-88

FACILITY PLAN - NOT FOR CONSTRUCTION

FIGURE 10-16
11TH AVE DROP SHAFT - 3D VIEWS

VAULT SERIAL NO.	DATE	MARK	NATURE REVISIONS	MADE	CHK'D	REV'D

Brown AND Caldwell

McMILLEN JACOBS ASSOCIATES

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NANCY LOCKE
DEPARTMENT OF FINANCE & ADMINISTRATIVE SERVICES
SEATTLE, WASHINGTON 20
BY:
PURCHASING & CONTRACTING SERVICES DIRECTOR

INITIALS AND DATE	INITIALS AND DATE
DESIGNED Designer CHECKED Checker	REVIEWED: DES. CONST. SDOT PROJ. MGR.
DRAWN Author CHECKED Checker	RECEIVED REVISED AS BUILT

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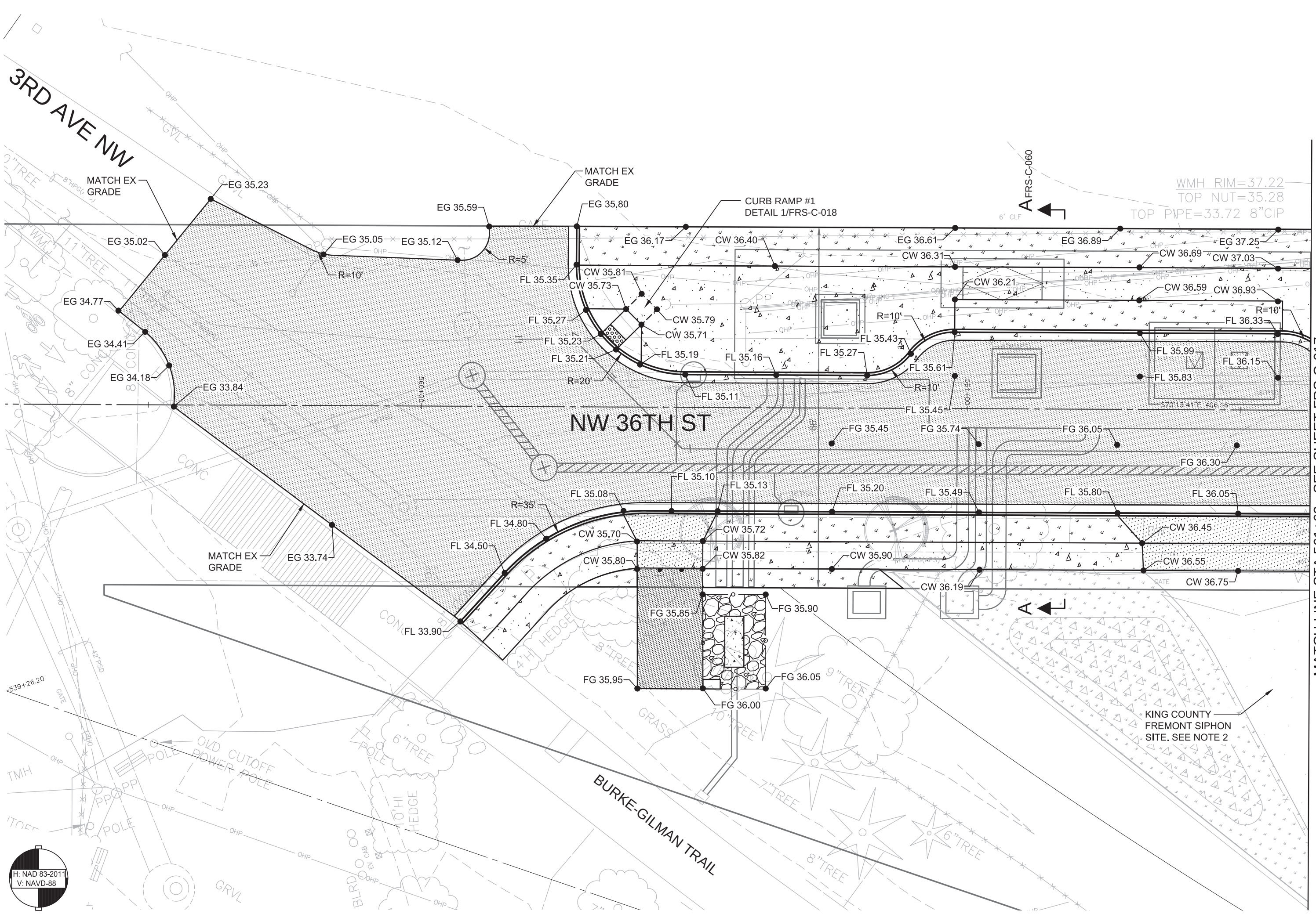
STATE OF WASHINGTON REGISTERED PROFESSIONAL ENGINEER

SHIP CANAL WATER QUALITY PROJECT

JOB NO. PC --- R/W --- CO ---
VAULT PLAN NO.
SHEET OF



NOTES:

1. VERIFY EX GR AT SITE LIMITS PRIOR TO FINAL SITE RESTORATION & NOTIFY ENGINEER OF ANY DISCREPENCIES.
2. NOTE TO REVIEWER: KING COUNTY FREMONT SIPHON PROJECT CURRENTLY UNDER CONSTRUCTION. PROPOSED SITE DESIGN CURRENTLY SHOWN. FINAL AS-BUILTS TO BE SURVEYED AND UPDATED AS DESIGN PROGRESSES.
3. LIMITS OF SITE RESTORATION/PHASING DEFINED WITHIN SPECIFICATIONS SECTION 01 11 00.
4. FINAL GRADING SHOWN FOR THE ENTIRE PROJECT. CONTRACTOR TO PHASE RESTORATION BASED ON CONSTRUCTION LIMITS AS DEFINED WITHIN SPECIFICATIONS SECTION 01 11 00.

PAVING LEGEND:

- TYPE [401_ OR 402_] ASPH PAVEMENT
- TYPE [401_ OR 402_] CONC PAVEMENT
- TYPE 410B CURB & GUTTER
- TYPE 410C CURB
- TYPE 420 SIDEWALK
- TYPE 430A DRIVEWAY

FIGURE 10-17
NORTH 3RD AVENUE / 174 - FINISHED SITE PLAN

FACILITY PLAN - NOT FOR CONSTRUCTION

DCG civil structural | McMILLEN JACOBS ASSOCIATES

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SEATTLE, WASHINGTON . . . . 20 . .
BY, ...
CITY PURCHASING & CONTRACTING SERVICES DIRECTOR

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DESIGNED ES
CHECKED JW, DS
DRAWN ES
CHECKED ED

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REVIEWED:
DES. CONST.
SDOT PROJ. MGR.
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ENGINEER'S NAME — STATE OF WASHINGTON — REGISTERED PROFESSIONAL ENGINEER

SHIP CANAL WATER QUALITY PROJECT

JOB NO. PC R/W CO
VAULT PLAN NO.
SHEET OF 428

VAULT SERIAL NO. | DATE | MARK | NATURE REVISIONS | MADE | CHKD | REVD

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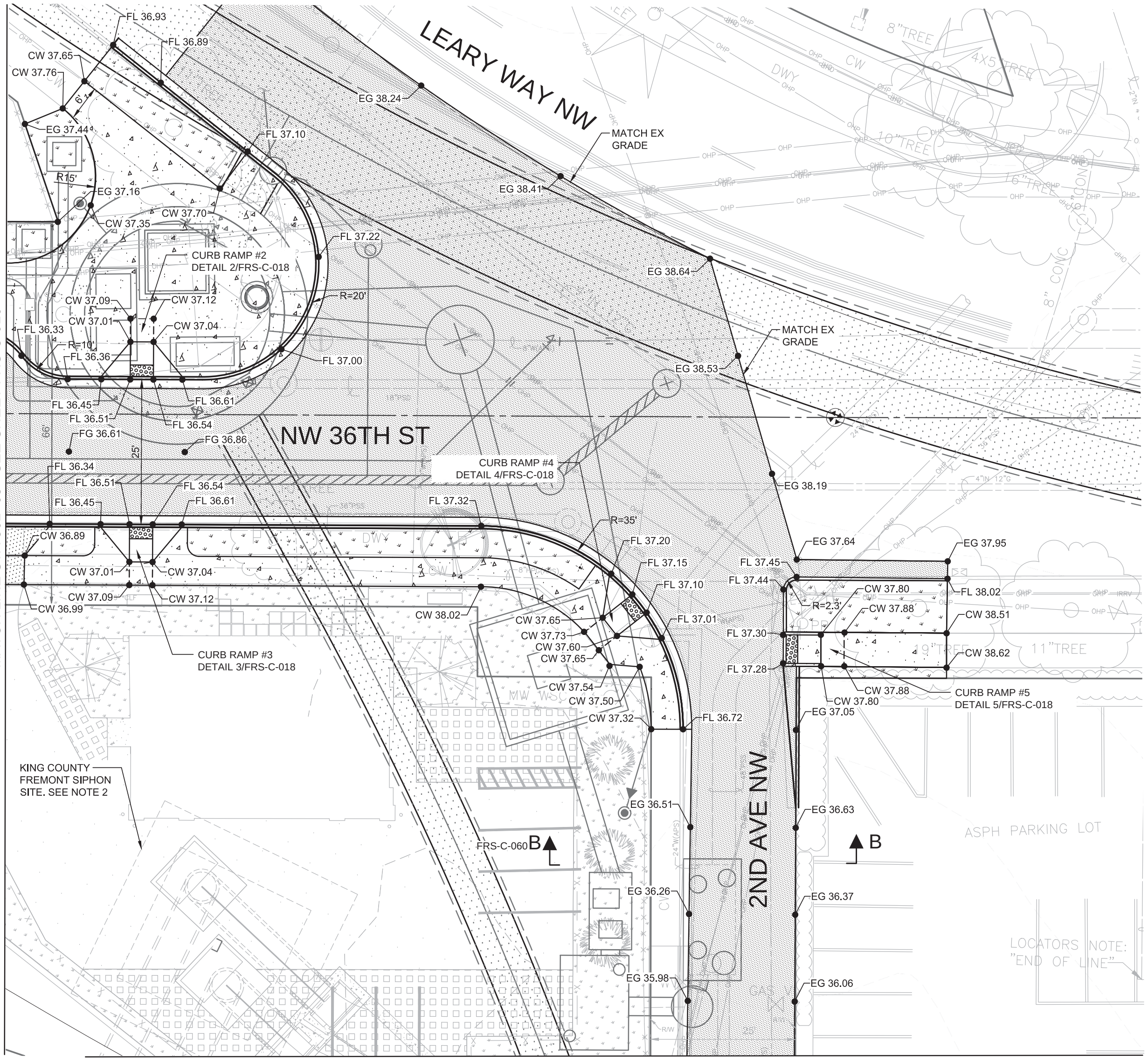
Pete Jacobchuk Oct-30-16 2:05pm



SCALE IN FEET

NOTES:

1. VERIFY EX GR AT SITE LIMITS PRIOR TO FINAL SITE RESTORATION & NOTIFY ENGINEER OF ANY DISCREPENCIES.
2. NOTE TO REVIEWER: KING COUNTY FREMONT SIPHON PROJECT CURRENTLY UNDER CONSTRUCTION. PROPOSED SITE DESIGN CURRENTLY SHOWN. FINAL AS-BUILTS TO BE SURVEYED AND UPDATED AS DESIGN PROGRESSES.
3. LIMITS OF SITE RESTORATION/PHASING DEFINED WITHIN SPECIFICATIONS SECTION 01 11 00.
4. FINAL GRADING SHOWN FOR THE ENTIRE PROJECT. CONTRACTOR TO PHASE RESTORATION BASED ON CONSTRUCTION LIMITS AS DEFINED WITHIN SPECIFICATIONS SECTION 01 11 00.

PAVING LEGEND:

- TYPE [401_ OR 402_] ASPH PAVEMENT
- TYPE [401_ OR 402_] CONC PAVEMENT
- TYPE 410B CURB & GUTTER
- TYPE 410C CURB
- TYPE 420 SIDEWALK
- TYPE 430A DRIVEWAY

MATCH LINE SEE THIS SHEET LEFT

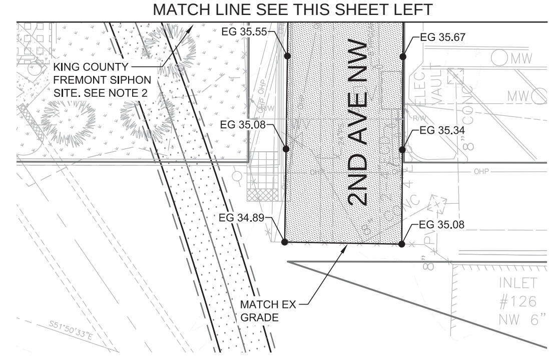


FIGURE 10-18
NORTH 3RD AVENUE / 174 - FINISHED SITE PLAN

FACILITY PLAN - NOT FOR CONSTRUCTION

DCG civil structural | McMILLEN JACOBS ASSOCIATES

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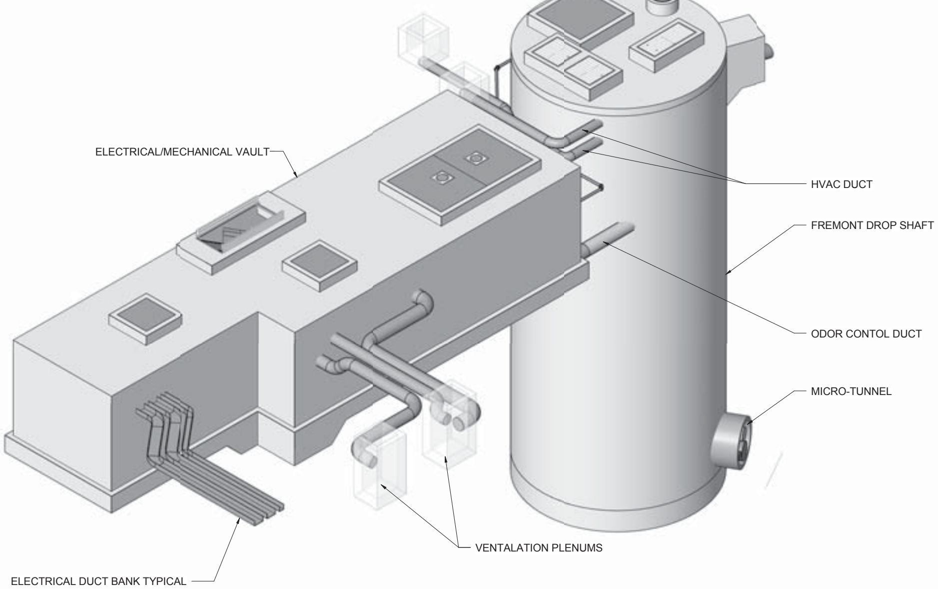

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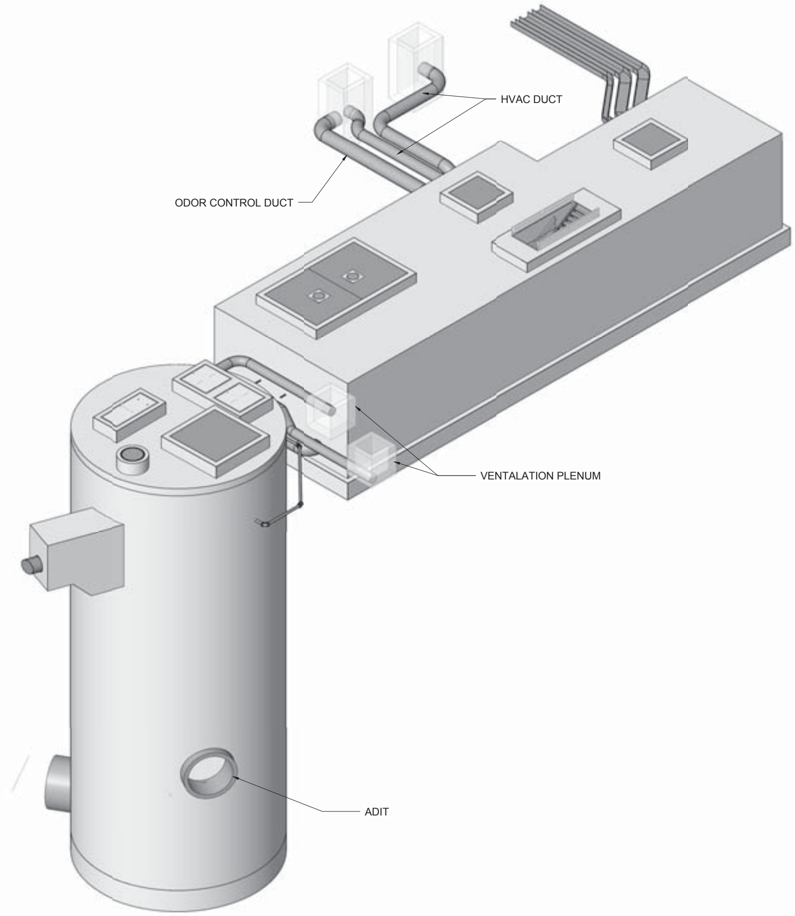
SCALE: NONE



YARD PIPING - 3D VIEW 2
SCALE: NONE

H:NAD 83/91
V:NAVD-88

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FIGURE 10-19
NORTH 3RD AVENUE / 174 - 3D VIEWS

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CO ---
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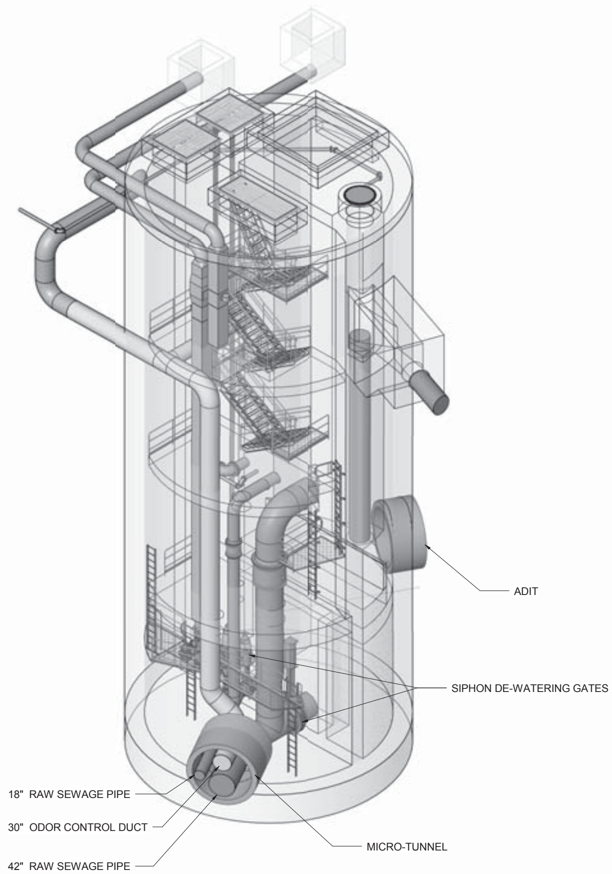
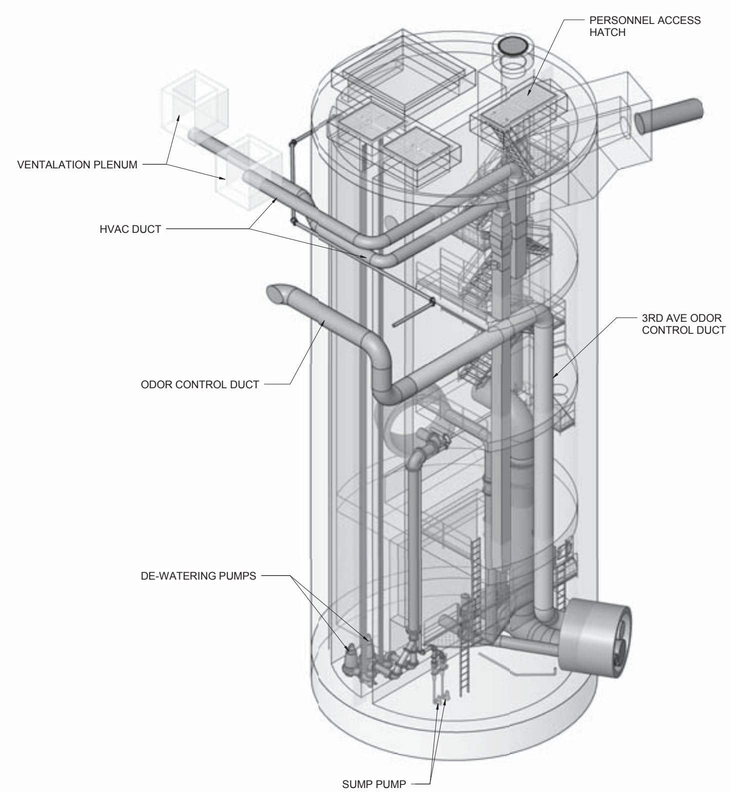


SHAFT - 3D VIEW 1

SCALE: NONE



SHAFT - 3D VIEW 2

SCALE: NONE

H:NAD 83/91
V:NAVD-88

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FIGURE 10-20

NORTH 3RD AVENUE / 174 - 3D VIEWS

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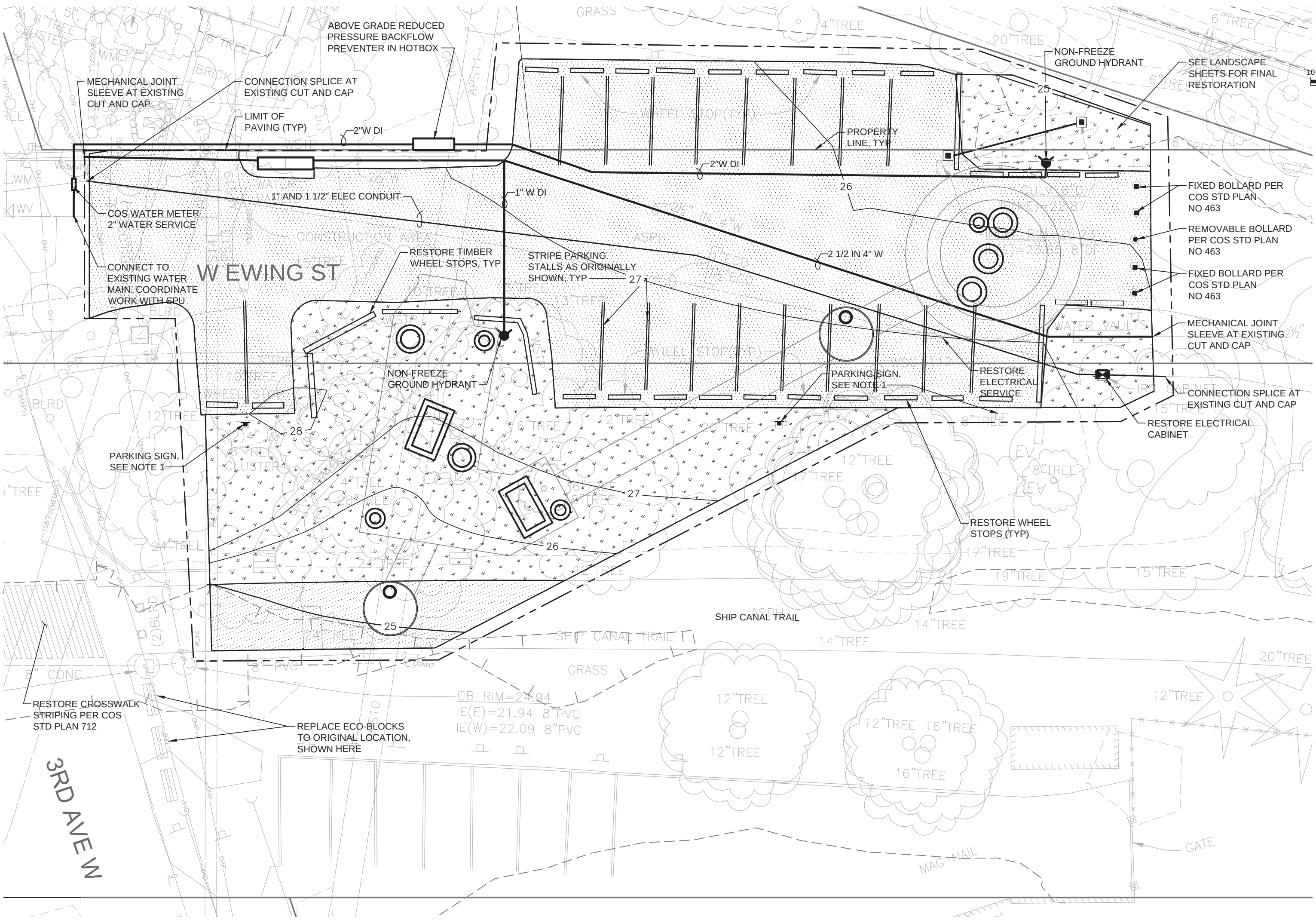

NOTES:

1. PARKING SIGNAGE TO BE COORDINATED WITH SDOT AND SEATTLE PARKS DEPARTMENT PRIOR TO FINAL DESIGN.
2. CONTRACTOR TO FIELD VERIFY AND LOCATE ALL UTILITIES PRIOR TO BEGINNING WORK AND NOTIFY ENGINEER OF ANY DISCREPANCIES.
3. RESTORE AND RE-OPEN THE SHIP CANAL TRAIL BEFORE COMPLETING OTHER SITE RESTORATION, SEE 3RS-C-010
4. FOR LANDSCAPE PLANS SEE 3RS-L-002.
5. FOR STORM DRAINAGE PLAN SEE 3RS-C-016.
6. SEE 3RD AVE SHAFT SITE -STRUCTURAL FOR ADDITIONAL INFORMATION.
7. SECANT PILE WALL AROUND SHAFT TO BE CUT OFF BELOW GRADE TO ALLOW UTILITIES SPACE TO ROUTE OVER.

PAVING LEGEND:

TYPE 402C ASPH PAVEMENT, 6" THICKNESS

LANDSCAPE RESTORATION AREA

FACILITY PLAN - NOT FOR CONSTRUCTION

FIGURE 10-21
SOUTH 3RD AVENUE DROP SHAFT - FINISHED SITE PLAN

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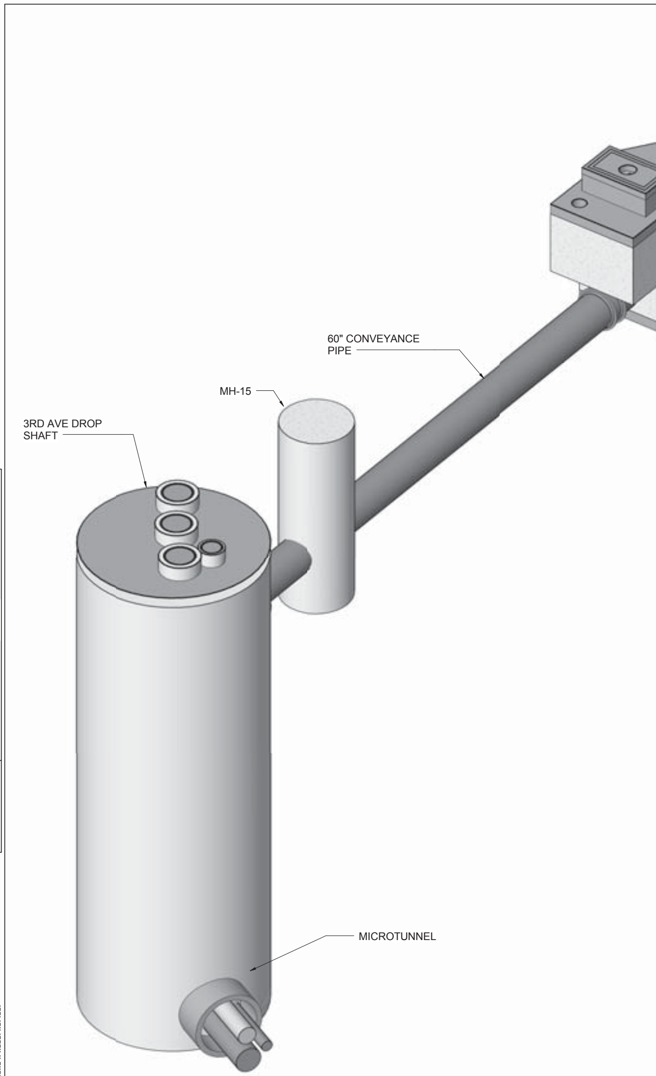
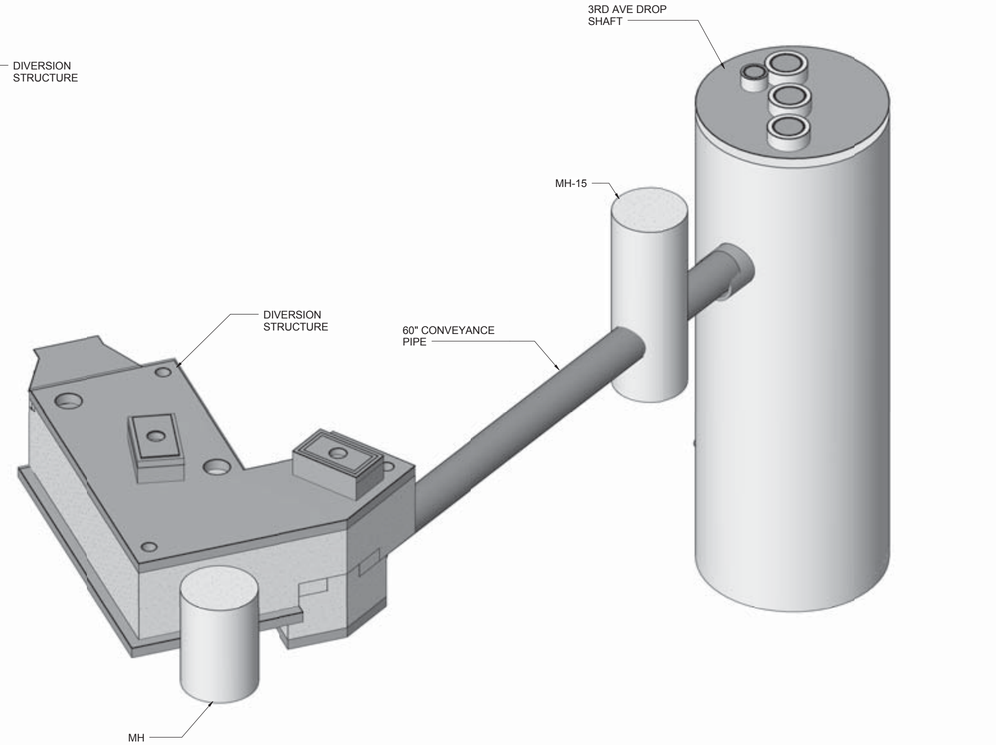


YARD PIPING - 3D VIEW 1
SCALE: NONE



YARD PIPING - 3D VIEW 2
SCALE: NONE

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FIGURE 10-22
SOUTH 3RD AVENUE DROP SHAFT - 3D VIEWS

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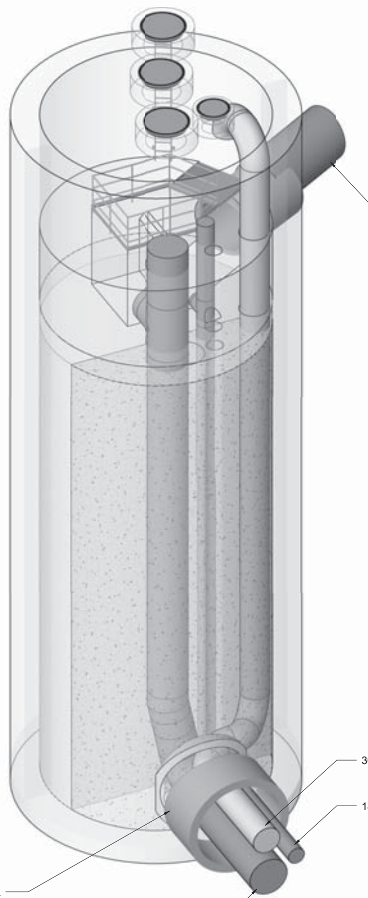
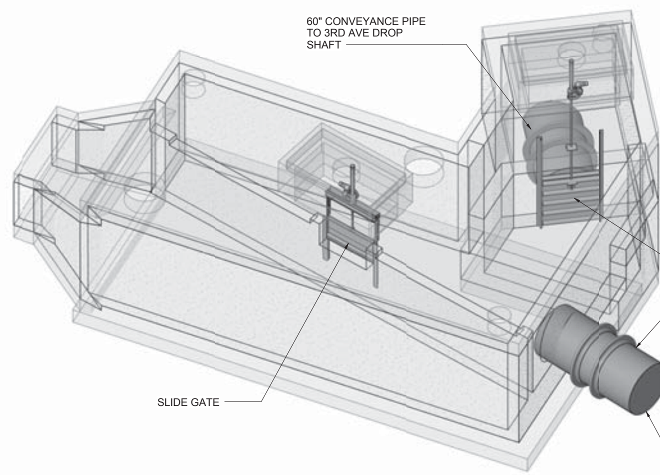


SHAFT - 3D VIEW 1
SCALE: NONE



DIVERSION STRUCTURE - 3D VIEW 2
SCALE: NONE

H:NAD 83/91
V:NAVD-88

FACILITY PLAN - NOT FOR CONSTRUCTION

FIGURE 10-23
SOUTH 3RD AVENUE DROP SHAFT - 3D VIEWS

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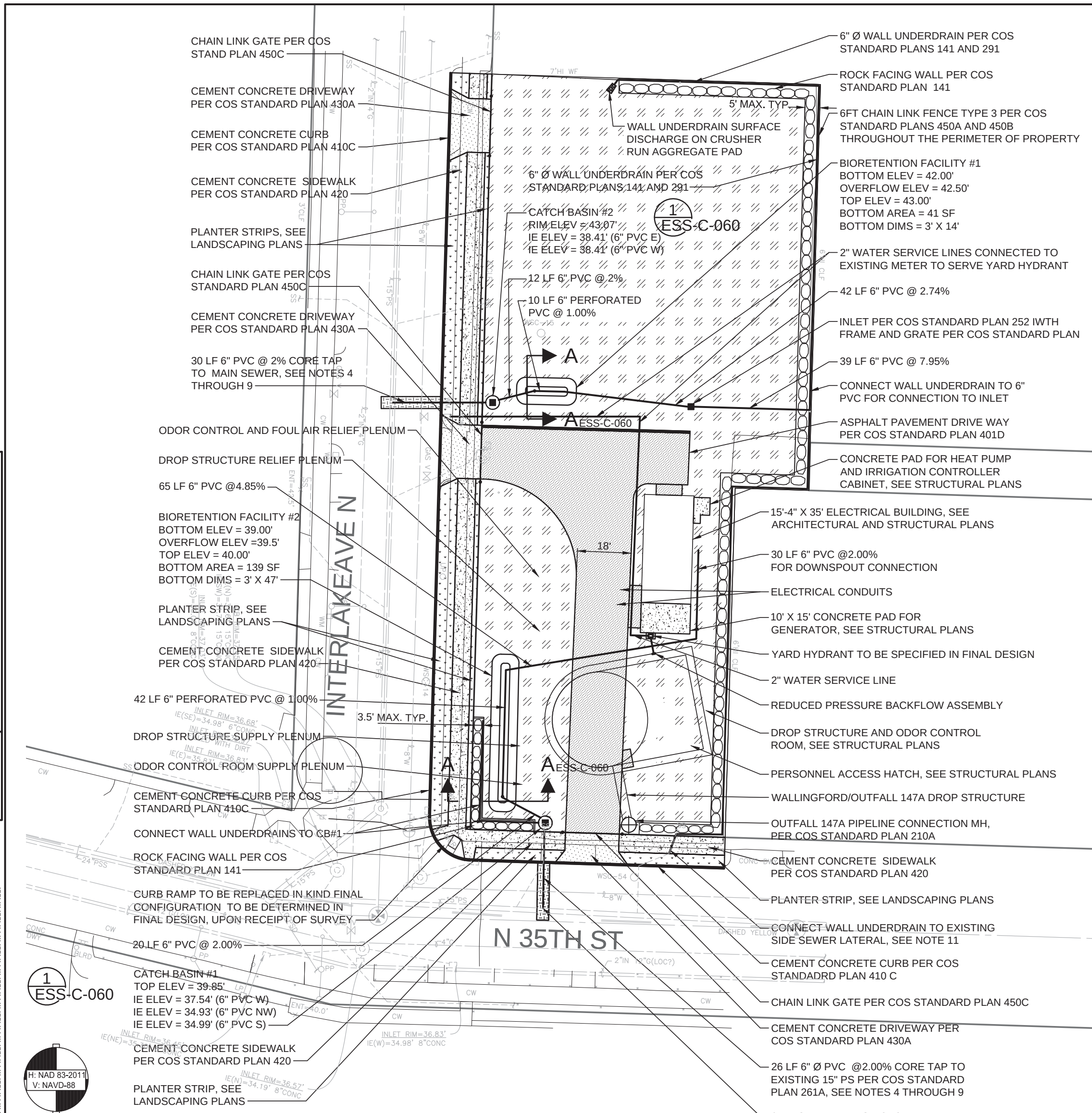

FACILITY PLAN - NOT FOR CONSTRUCTION

NOTES:

1. CONTRACTOR TO FIELD VERIFY AND LOCATE ALL UTILITIES PRIOR TO BEGINNING WORK AND NOTIFY ENGINEER OF ANY DISCREPANCIES.
2. POTENTIAL UTILITY CONFLICTS TO BE INVESTIGATED FURTHER IN FINAL DESIGN.
3. LIMITS OF SITE RESTORATION TO BE DETERMINED IN FINAL DESIGN.
4. ALL PIPE CONNECTIONS TO PUBLIC SEWER MAINS SHALL BE IN COMPLIANCE WITH SEATTLE MUNICIPAL CODE 21.6 AND IN ACCORDANCE WITH COS STANDARD PLAN NO. 261.
5. ALL PAVEMENT OPENINGS ON RIGHT-OF-WAY SHALL BE IN ACCORDANCE WITH COS STANDARD PLAN NO 404C AND COS STANDARD SPECIFICATIONS 2-02 AND 2-04.
6. PIPE TRENCH SECTIONS SHALL BE IN ACCORDANCE WITH COS STANDARD PLAN NO. 284.
7. PIPE BEDDING SHALL BE IN ACCORDANCE WITH COS STANDARD PLAN NO. 285.
8. PAVEMENT PATCHING SHALL BE IN ACCORDANCE WITH COS STANDARD PLAN NO. 404A.
9. ROADWAY PAVEMENT CONCRETE REPAIRS SHALL BE IN ACCORDANCE WITH COS STANDARD PLAN NO. 405A AND 405B.
10. LOCATION OF CROSSWALK PAVEMENT MARKINGS WILL BE DETERMINED IN FINAL DESIGN, AFTER CURB RAMP SURVEY IS AVAILABLE.

N

20 10 0 20 40

SCALE IN FEET

PROPOSED SITE AND UTILITIES LEGEND:

- CEMENT CONCRETE SIDEWALK
- CONCRETE CURB
- CEMENT CONCRETE DRIVEWAY
- PLANTING STRIP
- ASPHALT PAVEMENT
- ASPHALT BERM
- ROCK FACING WALL
- CHAIN LINK GATE
- INLET
- SEWER MAINTENANCE HOLE
- CATCH BASIN
- STORM DRAIN PIPE
- WATER METER AND SERVICE LINE
- SOIL AMENDMENT, SEE LANDSCAPING PLANS
- ROAD SURFACE RESTORATION

FIGURE 10-24

EAST PORTAL - FINISHED SITE PLAN

VAULT SERIAL NO.	DATE	MARK	NATURE REVISIONS	MADE	CHKD	REVD

C:\Users\pete jacobchuk\sharepoint\nc - cad\DOCUMENTCOS\TUNNEL\EAST SHAFT SITE\CIVIL\261-1064_P-ESS-C-020.dwg
Pete Jacobchuk Oct-19-16 1:32pm

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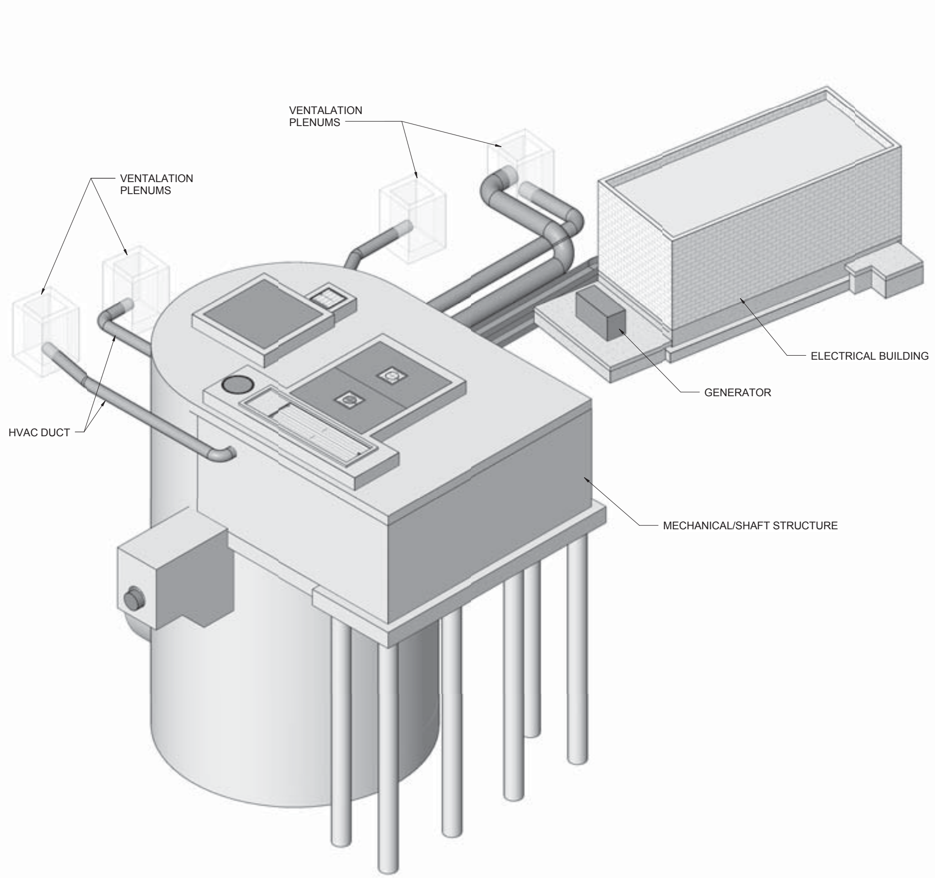

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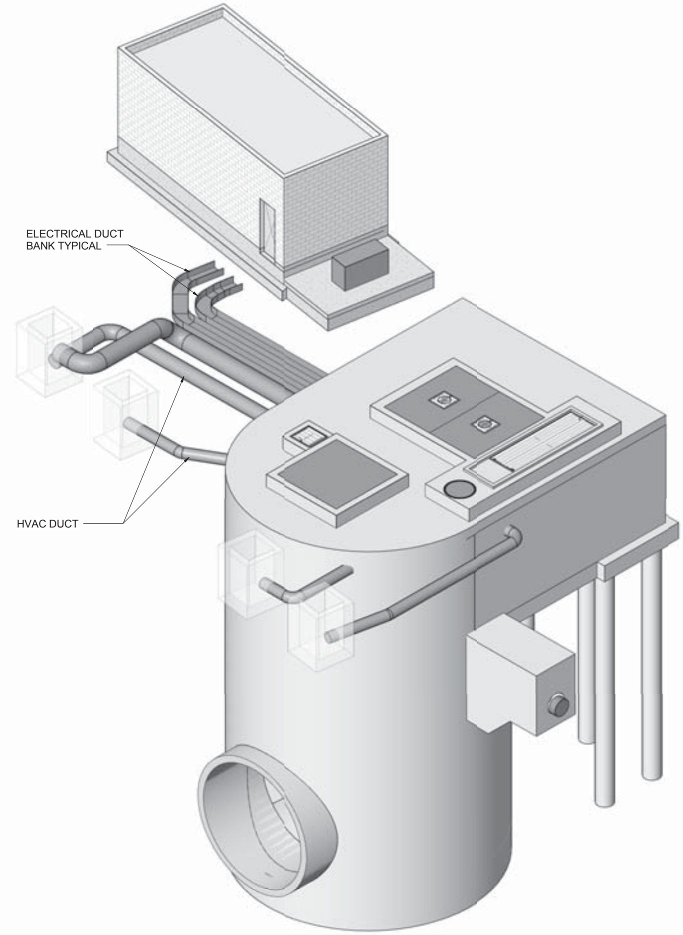
SCALE: NONE



YARD PIPING - 3D VIEW 2
SCALE: NONE

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			REVISIONS			

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FIGURE 10-25
EAST PORTAL - 3D VIEWS

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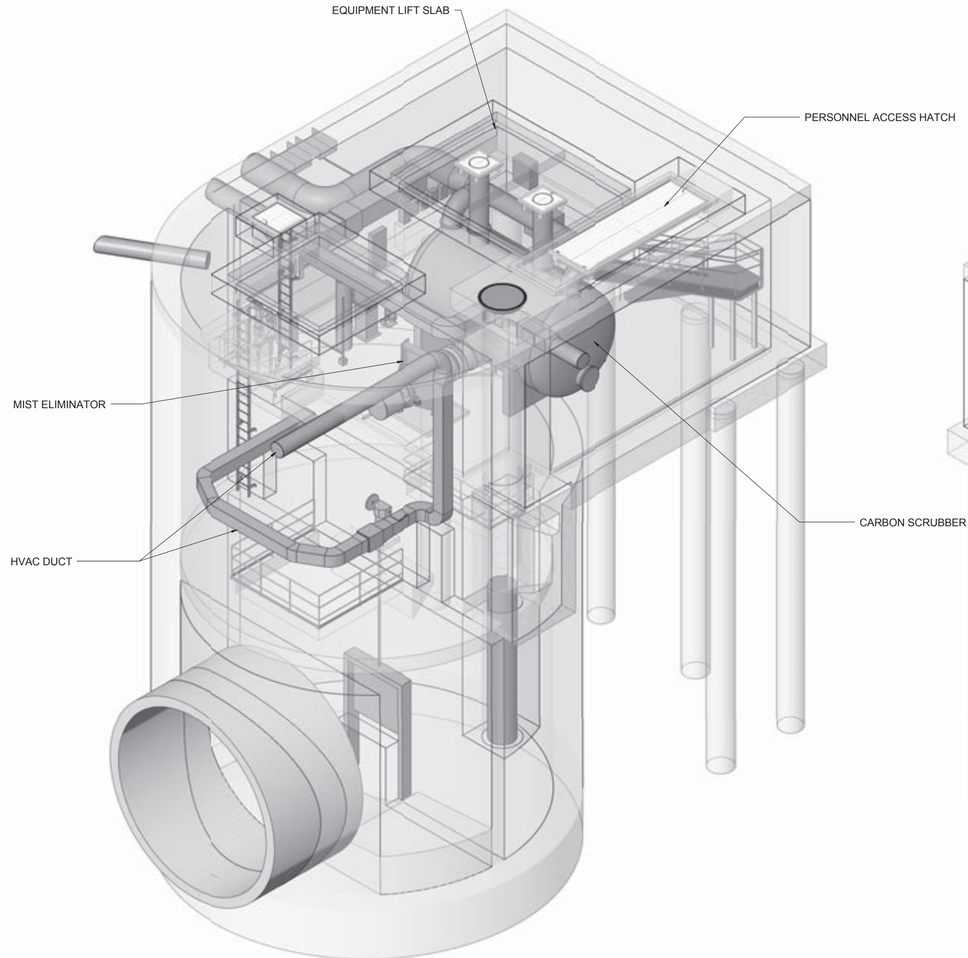
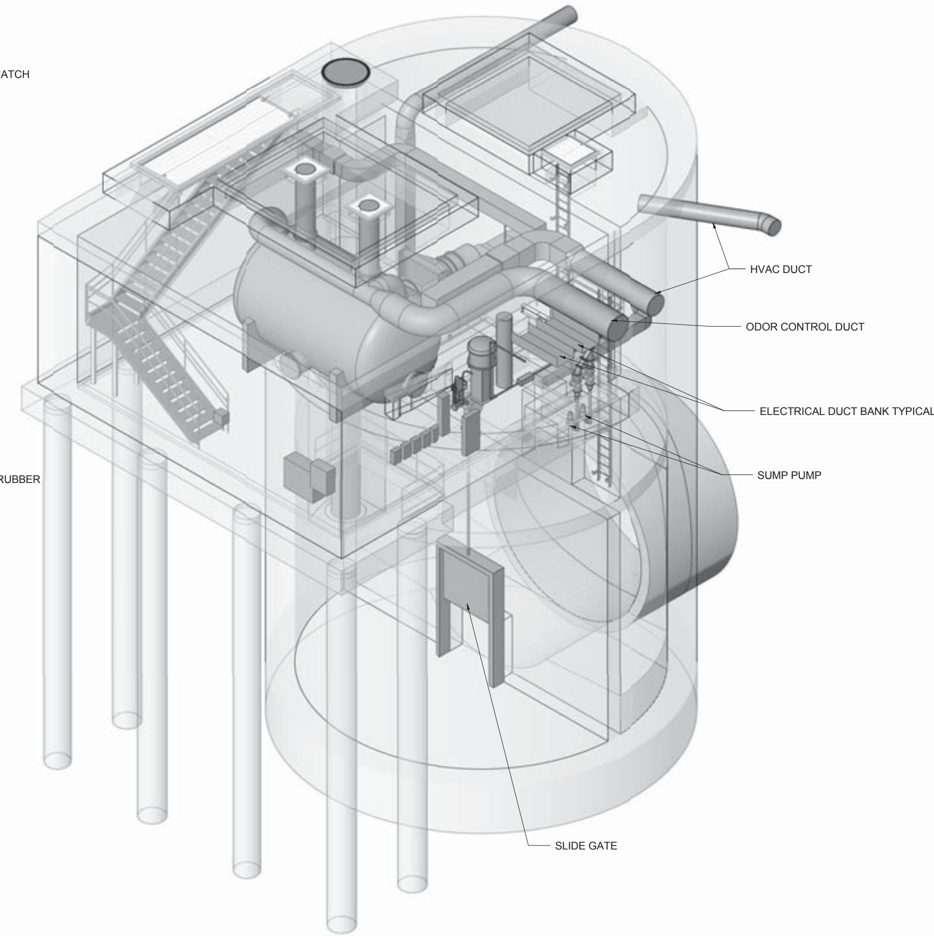


SHAFT - 3D VIEW 1

SCALE: NONE



SHAFT - 3D VIEW 2

SCALE: NONE

FIGURE 10-26
EAST PORTAL - 3D VIEWS

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Appendix D: Ship Canal Water Quality Project Integrated Modeling Report

Seattle Public Utilities | Protecting Seattle's Waterways

Ship Canal Water Quality Project Integrated Modeling Report

January 2018

Seattle Public Utilities
Ship Canal Water Quality Project
Integrated Modeling Report

January 2018

Prepared for:
Seattle Public Utilities
Seattle Municipal Tower, Suite 4900
700 Fifth Avenue
Seattle, Washington 98124-4018

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List of Abbreviations

Term	Definition
CSO	combined sewer overflow
DNRP	King County Department of Natural Resources and Parks
EPA	U.S. Environmental Protection Agency
GCM	general circulation model
IDF	intensity-duration-frequency
JPA	Joint Project Agreement
KCNI	King County North Interceptor
LTCP	Long-Term Control Plan
LWSC	Lake Washington Ship Canal
mgd	million gallons per day
MJA	McMillen Jacobs Associates
NAVD88	North American Vertical Datum of 1988
NGVD29	National Geodetic Vertical Datum of 1929
NIRR	no-impact release rate
NPDES	National Pollutant Discharge Elimination System
SCADA	supervisory control and data acquisition
SCWQP	Ship Canal Water Quality Project
SPU	Seattle Public Utilities
SWMM5	Stormwater Management Model
TEPS	tunnel effluent pump station
WPTP	West Point Treatment Plant

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Executive Summary

The City of Seattle and King County own and operate combined sewer systems that overflow at designated relief points during heavy rainfall events. The City and County are required by their respective National Pollutant Discharge Elimination System (NPDES) Permits and federal Consent Decrees to reduce Combined Sewer Overflows (CSO) to one uncontrolled overflow per outfall per year on a 20-year moving average. To help control CSOs, various storage and flow transfer concepts were evaluated in Seattle Public Utilities (SPU) *Plan to Protect Seattle's Waterways* and King County's Department of Natural Resources and Parks (DNRP) 2012 Long-term Combined Sewer Overflow Control Plan Amendment. The two agencies elected to implement a joint storage project to control CSOs along the Lake Washington Ship Canal (LWSC). This project, known as the Ship Canal Water Quality Project (SCWQP), consists of an approximately 14,000-foot-long, 18-foot 10-inch-diameter tunnel along the north side of LWSC from the Wallingford area of Seattle to the Ballard area. Figure ES-1 shows the extent of the SCWQP. As part of the SCWQP, the two agencies have embarked on a joint effort to create and calibrate a model that defines the operation of the system to confirm compliance with regulations. This Integrated Model is the subject of this report.

Purpose of the Integrated Modeling Report

This report describes the hydrologic and hydraulic modeling of the SCWQP storage tunnel integrated with the King County North Interceptor (KCNI) and associated inflows from the University of Washington area to the West Point Treatment Plant (WPTP). The purpose of this model is to confirm compliance with regulations and to guide final design of the joint project. The construction of this Integrated Model provides a common platform for both agencies to use in system assessment, provides updated models for the tunnel and allows the analyses to be extended through and beyond 2015, which is beyond the period used in the SPU LTCP. This model allows for a transition from the No-Impact-Release-Rates (NIRR) used in the SPU LTCP to control the discharge of stored CSO from the tunnel, to set point control based on level in the KCNI and at the Ballard siphon. This provides the same function as the NIRR and is more in line with expected future operational controls.

Modeling Analysis

The Integrated Model includes future flows and facilities projected to about the year 2035. Climate change projected rainfall was used to estimate flows at SPU outfalls, but not at King County outfalls. DNRP uses their approved long-term average of historic rainfall as a baseline for CSO facility sizing, while also considering the 20-year period of maximum actual CSO events. For the future condition, King County flows from the Matthews Park Pump Station (serving northeast Seattle and areas north of the city limits) were projected to the year 2060 to include its effect on performance. The model was used to simulate the integrated system using rainfall over the period of record from 1978 through 2015. The body of this report describes the process to develop and calibrate the Integrated Model, and the results of a 38-year simulation.

Figure ES-1. Extent of Ship Canal Water Quality Project Modeling

CSO Performance Modeling Results

A summary of the simulation results for SCWQP using the Integrated Model is shown in Table ES-1. Results are reported for the 20-year period from 1996 to 2015, which demonstrated the highest overflow frequencies in the full 38-year simulation. The results show an overflow frequency at all associated outfalls of well less than the regulatory requirement of one uncontrolled discharge per outfall on a 20 year moving average.

These results reflect a simulation in which the diversions of flow to the tunnel are controlled strictly on level of stored combined stormwater and wastewater.

Table ES-1. CSO Control Measures
20-year (1996–2015) Moving Average Annual Overflow Frequency Performance Results from Integrated Model Simulation

CSO Basin	Outfall Number	Average 20-year Overflow Frequency Existing Condition (per year)	Average 20-year Overflow Frequency Future Condition (2035)[a] (per year)
King County 11th Ave. NW	DSN 004	0.3	0.3
King County 3rd Ave. W	DSN 008	0.3	0.3
SPU Wallingford	147	0.3	0.4
SPU Fremont	174	0.3	0.4
SPU Ballard East	150/151	0.3	0.4
SPU Ballard West	152	0.3	0.4

Note: Results from model simulation assuming tunnel diversion gate closure based solely on the elevation in the storage tunnel without regard to control volumes over the period from 1996 to 2015.

a. Includes a climate change adjustment for SPU basins using a projected 2035 climate perturbed rainfall time series. DNRP has not applied an additional climate change adjustment, which is consistent with the County's approved LTCP. Also includes projected future (2060 planning horizon) King County flows from the Matthews Park Pump Station.

SECTION 1

Introduction

This report provides the process, methodology, and results of creating a hydrologic/hydraulic model combining the King County North Interceptor (KCNI) with the Seattle Public Utilities (SPU) Ship Canal Water Quality Project (SCWQP) tunnel and associated appurtenances. The SCWQP storage tunnel is designed to collect combined sewer overflows (CSOs) at King County- and SPU-permitted overflow sites along the Lake Washington Ship Canal (LWSC). The final model combining SPU and King County facilities is called the Integrated Model. This model extends and refines work that was begun in the SPU Long-Term Control Plan (LTCP) (Seattle Public Utilities, 2015a) providing a more accurate assessment of the performance of SCWQP for both SPU and DNRP. This work confirms the ability of SCWQP to meet regulatory requirements addressing the needs of both agencies. The information herein is similar to that found in the LTCP modeling reports (Seattle Public Utilities, 2012).

1.1 Purpose

The purpose of this effort is to understand the inter-related behavior of the SPU and King County systems including the SCWQP, the KCNI with its several other connections, and the King County West Point Treatment Plant (WPTP) to model regulatory compliance . Specific objectives to achieve this include the following:

1. Create and calibrate hydrologic/hydraulic models in a software platform usable by both agencies for the areas and structures contributing flow to both the KCNI and the SCWQP. The software platform chosen for this work is the 2016 version of the DHI MOUSE hydrology and hydraulics engine running under the MIKE URBAN interface (https://www.mikepoweredbydhi.com/products/mike-urban).
2. Create and calibrate a hydraulic model for the KCNI including all associated flow inputs, control structures, and control rules, and the WPTP influent control structure.
3. Create and calibrate a hydraulic model for the SCWCP including the combined sewage storage tunnel, tunnel effluent pump station (TEPS) returning stored flow to the KCNI under specified conditions, and diversion structures collecting CSO from existing overflow structures, diverting flow to the storage tunnel, and ultimately overflowing any excess flow after the tunnel is full to existing outfalls.
4. Project the effects of climate change on rainfall using available meteorological models to create revised time series of climate change adjusted rainfall through 2035 for application only to SPU CSO basins tributary to the SCWQP (CH2M, 2017).   DNRP

has not applied an additional climate change uncertainty factor. This is consistent with the County's approved LTCP.

5. Estimate uncertainties, other than climate change rainfall projections, associated with the calibration of the SPU CSO basin models in a manner identical to that used in the LTCP (Seattle Public Utilities, 2015b). SPU collaborated with DNRP to estimate a set of scaling factors unique to each SPU CSO basin. These scaling factors multiply the climate change adjusted rainfall time series to account for model and data uncertainties.

Once completed, the Integrated Model was used to simulate the performance of the integrated system over the 38-year period of rainfall from 1978 through 2015. The model is also being used to refine the design and operation of the diversion structures, and the operations of TEPS.

1.2 Background

The work reported here is a continuation of model analyses begun in the LTCP. As part of the LTCP, hydrologic/hydraulic models were developed in the U.S. Environmental Protection Agency's (EPA's) Stormwater Management Model (SWMM5) for the SPU CSO basins associated with the SCWQP (Seattle Public Utilities, 2012). These models were used to simulate the joint project storage tunnel included as an alternative in the LTCP (Seattle Public Utilities, 2015c). The SWMM5 tunnel model was further modified during preparation of the SCWQP Facility Plan (Seattle Public Utilities, 2017) refining the diversions into the tunnel. These models used a no-impact release rate (NIRR) time series provided by DNRP consisting of a continuous estimate of flows that could be discharged to drain the tunnel. The NIRR method of control was specified in the Joint Project Agreement (JPA) between SPU and DNRP (City of Seattle, 2016)

The previous model could not be extended beyond the end of 2009, the end of the NIRR and other boundary conditions provided by DNRP. It became evident that the period to which the models were simulated (through 2009) may not accurately represent basin behavior in subsequent wetter periods. It was thus decided that the models should be recalibrated using data from 2010 through 2015 as well as those collected in the SPU LTCP. The work described here was undertaken to facilitate future extension of the model analyses as new rainfall and operational observations were made. This work includes a conversion of the various models from their existing platforms to a common platform using the DHI MOUSE hydrologic/hydraulic engine running in the MIKE URBAN interface. With the integration of the models, the NIRR could be replaced, with a more operations-based control set point measurements scheme, and the tunnel operation could be controlled directly by measurements in the KCNI and at the Ballard siphon. This operation-focused methodology meets the intent of the NIRR and is more reflective of expected operation in the future.

1.3 Description of Report Contents

The remainder of this report describes the methodology for creation and calibration of the several parts that were eventually integrated into a single model of the system, the results derived from the model with adjustment of rainfall for climate change and other model uncertainties, and future refinements to the model.

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SECTION 2

Methodology

This section describes the development and calibration of the several sub-models used to create the Integrated Model.

2.1 Tributary Basins Developed and Calibrated

The KCNI included in the Integrated Model receives inflows from the following basins that were developed in MOUSE and calibrated against flow monitoring data. Figure 2-1 shows the basins involved.

- **King County University Regulator Basin:** Developed and calibrated by DNRP to support its CSO Plan Update as well as the Integrated Model. The output of this model is used as inflow to the Integrated Model.
- **King County Montlake Regulator Basin** including the SPU Leschi and Madison Valley basins: This model was originally developed and calibrated by Brown and Caldwell with subsequent refinement of the calibration by DNRP. This model supports the DNRP CSO Plan update as well as the Integrated Model. The output of this model is used as inflow to the Integrated Model.
- **King County Matthews Park Pump Station and Lake City Regulator:** This model includes the King County Lake City tunnel and regulator with tributary SPU basins. Inflow to the tunnel from the King County Matthews Park Pump Station is included. The Matthews Park Pump Station inflow was developed from other DNRP models.
- **East Wallingford:** This basin includes all inflow from SPU basins north of LWSC and east of the SPU Wallingford National Pollutant Discharge Elimination System (NPDES) 147A basin to the King County University Regulator. The model was developed and calibrated by DNRP.
- **SPU Wallingford Outfall 147 CSO Basins** with two overflow structures, designated 147A and 147B. The model was developed and calibrated by DNRP.
- **SPU Fremont Outfall 174 CSO basin.** The model was developed and calibrated by DNRP.
- **King County Central Trunk CSO Basins** extending upstream of the 3rd Avenue West Outfall to their Dexter Avenue Regulator, interconnection to the Elliott West Tunnel, and portions of Capitol Hill. The Capital Hill portion of the model was originally developed and calibrated by Brown and Caldwell for SPU and refined by DNRP. This was

combined with an existing DNRP model for the remainder of the system and then refined and recalibrated by DNRP.

- **SPU basins contributing flow downstream of the 3rd Avenue West overflow to WPTP:** This area includes the SPU Outfall 068 and Outfall 060 CSO basins as well as additional SPU inflows directly to the KCNI. Flow hydrographs from these were taken from other SPU and DNRP models.
- **King County Interbay Pump Station:** This includes all upstream areas tributary to the King County Elliott Bay Interceptor. Inflow hydrographs were provided by DNRP using other DNRP models.
- **King County 11th Avenue Northwest Outfall** including upstream SPU basins and the King County Carkeek Pump Station. The model was developed and calibrated by DNRP.
- **SPU Ballard Outfall 150/151 and Outfall 152 CSO Basins.** The models were developed and calibrated by DNRP.
- **King County Ballard Regulator basin** including SPU basins downstream of SPU Outfalls 150/151 and 152, and the King County 11th Avenue Northwest outfall. The model was developed and calibrated by DNRP.

Figure 2-1. Extent of the Integrated Model

2.2 Flow and Rainfall Monitoring Data

The calibration of the models used the flow and rainfall monitoring data collected by SPU during preparation of the LTCP (Seattle Public Utilities, 2010) and additional data collected following the LTCP period. The flow monitoring data collected in the post-LTCP period consisted of:

- SPU permanent monitors at overflow sites used to report overflows. These were part of the LTCP monitoring, and have continued to collect data
- Continued recording of precipitation at SPU and King County rain gauges
- Temporary and permanent monitors installed by DNRP to refine hydraulic calculations at the 11th Avenue Northwest, 3rd Avenue West, Montlake, and University overflow structures

2.3 Calibration and Review Methodology

Per agreement between the two agencies development, calibration, and review responsibilities were divided between SPU, DNRP, and the tunnel design consultant, McMillen Jacobs Associates (MJA).

Calibration of the hydrological parameters in MOUSE was facilitated by use of the PEST software (http://pesthomepage.org/Home.php) providing parameter estimation and uncertainty analysis. Under specified restraints, PEST runs the MOUSE RDI hydrology module multiple times, changing parameters until a best fit of model output to measured flow data is achieved.

A review process was undertaken as each system sub-model was calibrated. This involved a presentation by the modeler to all parties that discussed construction of the model, data sources, and results of the PEST calibration. Following this presentation, the agency or consultant not involved with the calibration undertook an independent review of the model and associated documentation to ensure that the model (1) was correctly constructed, (2) was documented sufficiently to allow future modelers to use it with confidence, and (3) met the accuracy expectations of the project. The reviews were conducted using a template created by SPU containing a comprehensive list of checks to be made.

The results of the reviews were submitted to the original modeler for comment and correction if necessary. The responses and corrections were subjected to a second round of review prior to final acceptance of the model. The individual model calibrations are discussed in the following sections.

2.4 Accounting for Climate Change and Other Uncertainties

In preparation of the SPU LTCP, a climate change adjustment to current rainfall was made, increasing historical rainfall by a constant 6 percent (Seattle Public Utilities, 2015b). This approach was updated for the current work using projections from an ensemble of available general circulation models (GCMs). The adjustments, known as perturbations, from these models result in a general increase in winter month precipitation and a reduction in summer precipitation. In addition, the models show that intense storms will become more intense and occur more frequently. The perturbations applied include an increase or decrease in monthly total precipitation, and an increase in the intensity of events of 2-year recurrence or rarer according to an intensity-duration-frequency (IDF) approach (CH2M, 2017).

Projections were made for the years 2035 and 2100 using a mean of the GCM models assuming a high atmospheric green house gas concentration. The mean annual increase in precipitation was found to be +3.5 percent for 2035 and +12 to +13 percent for 2100. The Integrated Model was run with the mean 2035 perturbed rainfall (modified for other model uncertainties as described below) to the SPU CSO basins that contribute flow to SCWQP. This result thus is representative of performance expected when the SCWQP is expected to be 10 years into its operation. Adjustments for climate change were not included for the DNRP basins in the model.

Other model uncertainties include factors to assess (1) the representativeness of the existing rainfall record used for model calibration, (2) the goodness of fit of model predictions to measured flow data, (3) the number and quality of flow measurements, and (4) hydraulic factors that introduce uncertainty in the results. SPU assessed these factors using procedures identical to those used in the LTCP (Seattle Public Utilities, 2015b) for the newly calibrated SPU CSO basin models. Scaling factors were derived for each SPU CSO basin that are used to multiply (scale) the 2035 perturbed rainfall time series.

SECTION 3

Individual Model Calibrations

This section discusses the construction and calibration of the various models that were eventually combined into the Integrated Model.

3.1 SPU Wallingford Outfall 147

DNRP provided the construction and calibration of this model under agreement with SPU. The basin includes two separate overflow structures, designated as 147A and 147B, which were calibrated separately.

The models included features from the previous SPU LTCP models to describe the head-flow relationship between the 147A overflow structure and KCNI, and the orifice connecting the 147B basin to KCNI (Seattle Public Utilities, 2012).

The hydrology of the basins was calibrated with PEST using flow measurements collected at the inflow to the overflow structures by SPU permanent monitors used to assess overflows. Hydraulic behavior of the structures was calibrated by reference to depth measurements in the overflow structure and flow measurements in the outfall conduits. Calibration was conducted using measurements from the SPU permanent monitors at the overflow sites over the period from 2008 through 2015, accounting for the SPU CSO Retrofit weir raising that occurred in 2010.

The results of the calibrations together with details of the models are presented in the DNRP report contained in Appendix A.

3.2 SPU Fremont Outfall 174

DNRP provided the construction and calibration of this model under agreement with SPU. The hydrology of the basin was calibrated with PEST using flow measurements collected at the inflow to the overflow structure by the SPU permanent monitor used to assess overflows. Hydraulic behavior of the structure was calibrated by reference to depth measurements in the overflow structure and flow measurements in the outfall conduits. Calibration was conducted using measurements over the period from 2008 through 2015, accounting for the SPU CSO Retrofit weir raising that occurred in 2010.

The results of the calibration together with details of the model are presented in the DNRP report contained in Appendix B.

3.3 SPU Ballard Outfalls 150/151 and 152

These two CSO basins are tributary to the King County Ballard Regulator. The regulator discharges to KCNI via the Ballard Siphon. The behavior of the two overflow structures is affected by the regulator operation and interact among themselves. As a result, the model includes both overflow structures, the Ballard Regulator and the interconnecting conduits leading to the Ballard Regulator. The model also includes SPU areas between the overflow structures and the regulator, and inflow from the 11th Avenue Northwest model discussed below.

Calibration of the hydrology of sub-basins upstream of the overflow structure was conducted at meters installed for the LTCP collecting data from 2008 through 2009. Final calibrations were conducted on flow measurements collected by SPU permanent monitors at the overflow structures covering the 2008 through 2015 period. Hydraulic calibration of the overflow structure behavior relied on measurements of depth in the overflow structures from the permanent meters installed to assess overflows. SPU CSO Retrofit weir raising that occurred in 2010 was accounted for.

The tributary areas between the overflow structures and the regulator were calibrated using supervisory control and data acquisition (SCADA) measurements at the regulator. The results of the calibration together with details of the model are presented in the DNRP report contained in Appendix C.

3.4 King County 11th Avenue Northwest Outfall

This model includes the King County Carkeek Pump Station and its area tributary for which a separate model was created and calibrated by DNRP. The Carkeek Pump Station model hydrology was calibrated to flow measurements from flow meters installed as part of the LTCP as well as DNRP's SCADA measurements at the pump station.

Hydrology calibration of the area between the Carkeek Pump Station and the overflow structure used data from flow meters installed as part of the LTCP as well as measurements made by the permanent CSO monitor installed by DNRP at the overflow structure.

The initial calibration was not considered adequate. To optimize the simulation of the overflow structure, DNRP subsequently added temporary flow monitors downstream of the overflow structure in the conduit leading to the Ballard Regulator, and in the outfall in 2017. Re-calibration was performed using data from these monitors. The results of the calibration together with details of the model are presented in the DNRP report contained in Appendix D.

3.5 King County Central Trunk and 3rd Avenue West Outfall

DNRP refined a preexisting model for these facilities leading to its 3rd Avenue West overflow structure. Data sources included DNRP measurements associated with the diversion to the Elliott West tunnel, the Dexter Regulator Station, a flow monitor just upstream of the 3rd Avenue West overflow structure installed as part of the SPU LTCP, other monitors installed as part of the SPU LTCP upstream of the Elliott West facilities, the permanent King County CSO monitor at the outfall, and additional temporary monitors around the overflow structure installed by DNRP to refine the hydraulic simulation. The calibration is discussed in Appendix E.

3.6 King County University and Montlake CSO Basins

DNRP refined preexisting models for these facilities as part of ongoing work to update its CSO Plan. The Integrated Model includes DNRP's anticipated storage projects near both the University and Montlake regulators.

3.7 King County Matthews Park and Interbay Pump Stations

DNRP refined preexisting models for the tributary areas to these pump stations. Matthews Park includes SPU basins along Thornton Creek. Flow from separated areas north of Seattle city limits is also included. A Matthews Park model was constructed for both the current condition and for projected conditions in 2060. The future-condition model includes projected increases in population and assumed increases in infiltration/inflow due to sewer degradation included in DNRP planning. The output of these models is used as inflow to the Integrated Model at the North Portal of the Lake City tunnel.

The Interbay Pump Station receives flow from the DNRP Elliott Bay Interceptor including areas of Seattle south of the pump station to the Norfolk Regulator. The output of the Interbay Pump Station simulation, using other DNRP models, is used as inflow to the Integrated Model.

3.8 King County North Interceptor

SPU assembled a MOUSE model of the existing KCNI from preexisting models and drawings from the SPU and DNRP plan libraries. DNRP then completed the model construction and calibrated its response to flow monitoring data collected during the SPU LTCP (Seattle Public Utilities, 2010), DNRP flow monitoring data, DNRP SCADA measurements, and field observations by DNRP staff. The model was tuned to match level measurements by adjusting minor losses, sediment depths, and friction losses. Flows at key locations including overflows were then optimized. The calibration process and results are described in Appendix E.

3.9 SCWQP Tunnel Model Construction

The construction of the SCWQP Tunnel model by the MJA team progressed through several stages to be suitable for integration with the KCNI model. The original model construction was completed in EPA's SWMM5. SWMM5 was used because the early SCWQP modeling relied on LTCP models (Seattle Public Utilities, 2015c). This model represented the tunnel at an approximately 10 percent design level and relied on preliminary layouts for many of the facilities. The model included hydraulics, limited hydrology, and inflow time series to provide much of the inflow (inflow time series were generated by running upstream contributing CSO basin models and saving output at specific locations). The inflows represent flow generated in each of the CSO basins along the LWSC upstream of the existing overflow weirs and proposed diversions to the tunnel. Hydraulics included conveyance for the following:

- Existing hydraulics for CSO basins 152, 150/151, 174, 147A, and 147B for connectivity between inflow locations and proposed SCWQP structures. The Facility Plan model (originating from the LTCP models) modified for initial design calculations was used as the source of this information.
- Proposed SCWQP tunnel.
- Proposed SCWQP TEPS.
- Proposed 11th Avenue Northwest diversion.
- Proposed 3rd Avenue West diversion and associated siphon under LWSC.
- Proposed diversion and associated conveyance for CSO basin 174.
- Proposed diversion and associated conveyance for CSO basins 147A and 147B.
- Proposed control rules to operate all proposed facilities.

Boundary conditions were also included as necessary to dictate the levels at outfall locations for the LWSC and KCNI.

Once the decision was made to complete SCWQP modeling activities in the 2016 version of the DHI MOUSE hydrology and hydraulics engine running under the MIKE URBAN interface, the SWMM5 SCWQP Tunnel model was converted to that platform. Differences between the two platforms and DNRP modeling standards, were accounted for during model conversion. This included the following:

- **Loss coefficients at modeled nodes:** MIKE URBAN has allowances to enter loss coefficients at maintenance holes, which were not included in the SWMM5 model. They were added to the MIKE URBAN model upon conversion to conform to DNRP standards.

- **Control rule operations:** Each platform has specific settings required to correctly enter control rules to operate gates and pumps. The rules contained in the SWMM5 model were modified as necessary and input into MIKE URBAN.
- **Modeling of weir structures as orifices:** MIKE URBAN does not account for submersed weirs when modeling physical structures as weirs, while SWMM5 does. Therefore, all elements modeled as weirs in SWMM5 were modeled as orifices in MIKE URBAN. This SPU standard has been adopted by DNRP.
- **Model datum:** The SWMM5 model was constructed using the North American Vertical Datum of 1988 (NAVD88) as consistent with LTCP models and per SPU standards. DNRP uses the METRO datum, which is the National Geodetic Vertical Datum of 1929 (NGDV29) plus 100 feet. All hydraulic elements were converted from NAVD88 to METRO datum by adding a conversion of +96.42 feet. This made the model compatible with all other DNRP models.

In addition to the items listed above, continuity in long conduits is a concern in both SWMM5 and MIKE URBAN. This is further complicated by the Priessman slot used in the MOUSE computational engine. The tunnel conduits in SWMM5 were divided into shorter segments to reduce continuity errors. In MOUSE, the conduits representing the tunnel sections were modeled as long conduits and there was a concern that the model could potentially overestimate available storage volume. Simulations were run to ensure mass balance by comparing simulated tunnel inflow with actual tunnel volume (approximately 30 million gallons). To ensure continuity, the tunnel length was shortened, the Priessman slot width was adjusted, and computation grid settings (specifying internal computational points in the long conduits) were adjusted to ensure that the model stored no more volume than will be available.

The MJA consultant team undertook model conversion, and made a presentation to DNRP and SPU staff upon completion. After the presentation, DNRP and SPU conducted an independent review of the MIKE URBAN model and associated documentation. Suggested modifications were then incorporated.

The SCWQP Tunnel model continued to be revised as design progressed to the 30 percent and 60 percent milestones. These revisions impacted proposed diversion and conveyance structures for the following locations:

- SPU Ballard Outfalls 150/151 and 152
- SPU Fremont Outfall 174
- SPU Wallingford Outfall 147 including overflow structures 147A and 147B
- King County 11th Avenue Northwest Outfall
- King County 3rd Avenue West Outfall

- SCWQP TEPS delivering a nominal average of 32 mgd

Model revisions to inflow time series and model hydrology to incorporate newly calibrated CSO basin models as described above were also made. The SCWQP model was representative of 60 percent design for proposed facilities with an 18-foot 10-inch-diameter tunnel, included the most up-to-date inflows and hydrology to generate system flows, and relied upon the NIRR for downstream boundary conditions to allow for TEPS operation. This version of the model was carried forward to integration with the KCNI model.

3.10 Integration of King County North Interceptor with SCWQP Model (Integrated Model)

DNRP provided the MJA consultant team with the model of KCNI developed in MIKE URBAN, which was developed by SPU and DNRP modeling staff. This model includes all hydraulics for the KCNI from the Montlake and University regulators through the WPTP influent control structure. Proposed CSO storage facilities are included in this model version for the Montlake and University CSO basins. The recent modification to the DNRP Fremont Siphon is also included. Key inflow locations include the following:

- King County Montlake CSO basin flows from the South Lake Washington trunk
- King County University CSO basin flows
- King County Matthews Park Pump Station flows (model includes Lake City Tunnel and Regulator)
- King County flows in the Central Trunk upstream of the 3rd Avenue West overflow structure
- King County 11th Avenue Northwest flows including SPU basins upstream of the overflow structure
- King County Carkeek Pump Station flows
- King County Interbay Pump Station flows
- SPU Ballard CSO basin inflows to the overflow structures for Outfalls 150/151 and 152

This model was merged with the SCWQP Tunnel model to create one model of KCNI and proposed SCWQP tunnel and related facilities. This model is known as the Integrated Model. The hydraulics related to the SCWQP tunnel were implemented in the model as necessary to represent the proposed facilities. The model also includes the basin hydrology (sub-catchments) for SPU Fremont Outfall 174, SPU Wallingford overflow structures 147A and 147B, and portions of SPU Ballard Outfalls 150/151 and 152, which are downstream of the location where the inflow time series for those basins is applied.

The Integrated Model retains the inflow time series requirements found in the individual KCNI and SCWQP models; however, it replaces the need of the NIRR because it simulates flows and levels in KCNI that are used to dictate TEPS operation. This provides greater accuracy and extendibility in that the Integrated Model is now used to directly control the TEPS discharge using the simulated depth in the KCNI as a surrogate for flow to the WPTP. Discharge from TEPS is allowed only after flows into WPTP fall below 250 million gallons per day (mgd), the same concept used to develop the earlier NIRR. The calibration of KCNI and inclusion of proposed storage facilities at the University and Montlake regulators improves the ability to define allowable flow from TEPS in the future when these facilities are constructed. Figure 3.1 shows the extent of the Integrated Model.

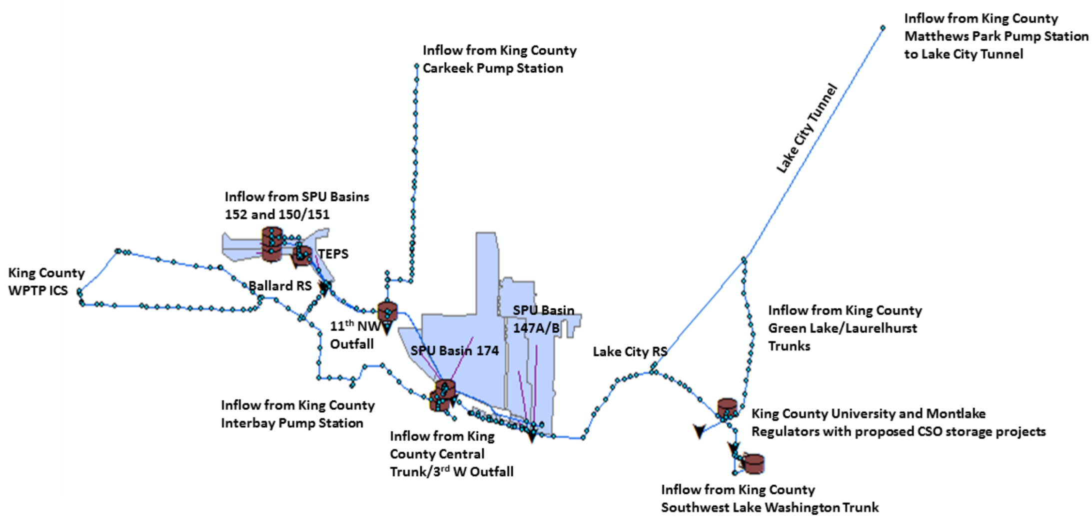

Figure 3.1. Integrated Model components

The consultant team provided DNRP with the Integrated Model for further review and refinement. DNRP modelers revised KCNI hydraulics based on continued calibration work, programmed in the control rules to ensure intended operation, and reviewed the model to ensure conformance with DNRP standard modeling protocols. The primary revisions included refining operations of the proposed University and Montlake CSO facilities, adjusting Manning's roughness coefficients in KCNI pipes, and adding the control rules for the drain and solids gates and for the solids and drain pumps in the TEPS.

The resultant version of the Integrated Model relies on real-time control logic to dictate diversion structure and TEPS operation. Logic specifying the operation of diversion gates that direct flow to the tunnel are strictly based on tunnel level. Each tunnel diversion gate closes once the tunnel reaches a specified elevation at TEPS. Once tunnel diversion gates close, a CSO event will occur if flows continue to rise in the diversion structure. The diversion gates are reopened once the level at TEPS has fallen to a specified set point as the tunnel is drained.

Startup of TEPS pumps is allowed based primarily on the level in the KCNI, which correlates to flow at WPTP. TEPS discharges to the King County Ballard Wet Weather Siphon via a control gate operated so as to not impact overflows at the King County Ballard Regulator or exceed allowable flows to the WPTP.

SECTION 4

SCWQP Performance Modeling Results

This section presents the determination of revised control volumes for each outfall based on the development and calibration of individual basins as discussed in Section 3, and presents the results of an Integrated Model simulation.

4.1 Control Volumes

DNRP ran the individual basin models under assumptions similar to those used in the SPU LTCP (Seattle Public Utilities 2015c). For SPU basins tributary to the tunnel, scaling factors including a 6 percent increase for climate change and a 1.0 to 1.5 percent increase for other uncertainties were included. No climate change or other uncertainty adjustments were included for DNRP, but the 2060 flow projections for the Matthews Park pump station were assumed. Similar to the SPU LTCP (Seattle Public Utilities 2015b), the control volume is defined as the volume that must be controlled to result in an average of one untreated overflow over the 38-year simulation (38th largest overflow volume). Table 4-1 presents the results of these simulations.

Table 4-1. Long-Term Modeling Results (1978 to 2015): Control Volumes

CSO Basin	Outfall	38-year Control Volume (MG)[a]
King County 11th Ave. NW	DSN 004	3.14
King County 3rd Ave. W	DSN 008	4.07
SPU Wallingford	147	2.36
SPU Fremont	174	1.56
SPU Ballard East	150/151	0.58
SPU Ballard West	152	6.55
	TOTAL	**18.3**

[a] *Volume that needs to be controlled to result in an average of one overflow per year over the 38-year simulation*

4.2 Results of Integrated Model Simulation

The Integrated Model was run for the period 1978 through 2015 assuming that all sluice gates allowing flow into the tunnel are controlled by the elevation of stored stormwater and wastewater in the SCWQP tunnel. A signal to close the gates occurs in the model when the simulated depth at TEPS reaches a specified set point elevation slightly below the soffit of the tunnel at the upstream end. The model was run both with current rainfall, and with the scaled 2035 rainfall for SPU basins described in Section 2.4. The existing Matthews Park pump station flows were used for the existing rainfall simulation, and the “future” (2060 projection) Matthews Park pump station flows were included for the 2035 simulation. The University and Montlake planned storage facilities were included in both model simulations.

Table 4-2 presents the frequency of overflows at each project outfall over the last 20 years of the model simulation (1996–2015), which exhibited the highest 20-year frequency over the total 38-year simulation. Results at SPU outfalls are presented for the simulated condition of control of diversions based on depth of combined stormwater and wastewater in the tunnel.

Table 4-3 presents the simulated overflow volumes over that last 20 years of simulation together with the peak rates of diversion reported for the full 38-year simulation. Overflows generally occur only during the largest events in the rainfall series.

Table 4-2. CSO Control Measures
20-year (1996–2015) Moving Average Annual Overflow Frequency Performance Results from Integrated Model Simulation

CSO Basin	Outfall Number	Average 20-year Overflow Frequency Existing Condition[a] (per year)	Average 20-year Overflow Frequency Future Condition (2035)[b] (per year)
King County 11th Ave. NW	DSN 004	0.3	0.3
King County 3rd Ave. W	DSN 008	0.3	0.3
SPU Wallingford	147	0.3	0.4
SPU Fremont	174	0.3	0.4
SPU Ballard East	150/151	0.3	0.4
SPU Ballard West	152	0.3	0.4

Note: Results from model simulation assuming diversion gate closure based solely on the elevation in the storage tunnel without regard to control volumes over the period from 1996 to 2015.

a. *Assumes existing rainfall and existing flows from the King County Matthews Park Pump Station*

b. *Includes a climate change adjustment for SPU basins using a projected 2035 climate perturbed rainfall time series. DNRP has not applied an additional climate change adjustment, which is consistent with the County's approved LTCP. Also includes projected future (2060 planning horizon) King County flows from the Matthews Park Pump Station.*

Table 4-3. CSO Control Measures
20-year (1996–2015) Moving Average Annual Overflow Volume Performance Results from Integrated Model Simulation

CSO Basin	Average 20-year Overflow Volume Existing Condition[a,c] (million gallons per year)	Average 20-year Overflow Volume Future Condition (2035)[b,c,d] (million gallons per year)	Peak Conveyance Design Flow rate (mgd)
King County 11th Ave. NW	2.1	2.3	168
King County 3rd Ave. W	2.0	2.4	103
SPU Wallingford Outfall 147	0.8	1.2	45
SPU Fremont Outfall 174	0.6	0.8	39
SPU Ballard Outfall 150/151	0.4	0.5	60
SPU Ballard Outfall 152	1.9	2.7	80

Note: Results from model simulation assuming diversion gate closure based solely on the elevation in the storage tunnel without regard to control volumes over the period from 1996 to 2015.

a. *Assumes existing rainfall and existing flows from the King County Matthews Park Pump Station*

b. *Includes a climate change adjustment for SPU basins using a projected 2035 climate perturbed rainfall time series. DNRP has not applied an additional climate change adjustment, which is consistent with the County's approved LTCP. Also includes projected future (2060 planning horizon) King County flows from the Matthews Park Pump Station.*

c. *Results from model simulation assuming diversion gate closure based solely on the elevation in the storage tunnel.*

d. *King County volumes increase in the future condition due to the inclusion of the 2060 projected flows from Matthews Park Pump Station and climate change modified flows from SPU.*

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SECTION 5

Future Changes

The Integrated Model results presented here are based on about a 60 percent design level for the diversion structures controlling flow to the SCWQP tunnel and TEPS. Design is currently moving forward with 90 percent diversion structure designs, with design completion in 2018. The suggested changes to the diversion structure optimizations have been examined and it has been determined that they would only marginally affect the overflow frequency results shown in Table 4-2. The Integrated Model will be used throughout the design optimization process to test options and provide guidance to the design teams for changes necessary to prevent significant changes in expected overflow frequency.

The Integrated Model will be updated as each phase of the SCWQP design is completed to ensure that compliance is maintained. A further round of update and calibration is scheduled after compliance monitoring data are gathered during the acceptance/commissioning phase following construction.

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SECTION 6

Conclusions

The Integrated Model described herein has been constructed, reviewed, and calibrated so that it provides an accurate simulation of expected system behavior under existing and potential future rainfall conditions. Simulations with the model over the 38-year period of rainfall record (1978–2015) indicate that average overflow frequencies after the project begins operations will be less than 0.5 per year averaged over the last 20 years, more than meeting the regulatory requirement of once per year on average.

Ongoing changes in facilities design have been examined and are not expected to more than marginally change the overflow frequencies presented in Table 4-2. Design changes will be incorporated in the Integrated Model as they are made for confirmation of performance and adjustment as necessary.

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SECTION 7

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Appendix A: Wallingford Basin 147 Mike Urban Model Development and Calibration Report

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Wallingford Basin 147 Mike Urban Model Development and Calibration Report

Hal Mullis

January, 2018

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Table of Figures and Tables

List of Abbreviations

CSO	Combined Sewer Overflow
CSS	Combined Sewer System
DHI	Danish Hydraulics Institute
DWF	Dry Weather Flow
FRC	Fast Response Component
GIS	Geographic Information System
KC	King County
MGD	Million gallons per day
MH	Manhole
NPDES	National Pollutant Discharge Elimination System
RDII	Rainfall-Dependent Infiltration and Inflow
RMSE	Root-Mean-Square Area
RTC	Real Time Control
SCADA	Supervisory Control And Data Acquisition
SCWQP	Ship Canal Water Quality Project
SPU	Seattle Public Utilities
SRC	Slow Response Component
WTD	Wastewater Treatment Division

1 Introduction

This report documents the development and calibration of hydrologic and hydraulic models for the City of Seattle's Combined Sewer System (CSS) in the Wallingford area (Basin 147) and its downstream connection to King County's North Interceptor. Basin 147 includes two sub-basins (147A and 147B) with a common overflow discharge outfall into Lake Union. Basin 147 averaged over 38 overflow events per year during the calibration period. Normal dry weather flows from Sub-Basins 147A and 147B discharge to the King County North Interceptor at separate connection points. SPU previously developed EPA SWMM5 hydrologic and hydraulic models as part of their 2015 LTCP. Information gained and lessons learned from that modeling effort were used in the current effort to develop and calibrate a model of the basin and overflow structure using the municipal wastewater modeling software MIKE URBAN, developed and distributed by the Danish Hydraulics Institute (DHI). MOUSE is the hydrologic and hydraulic engine within the Mike Urban software. The results of this modeling effort contributed to the development of an Integrated Model with the King County North Interceptor and will contribute to planning, design, construction and operation of the joint Ship Canal Water Quality Project.

1.1 Service Area and Operation

The study area for Basin 147 includes approximately 295 acres tributary to the overflow structures and an additional 19 acres downstream of the Sub-Basin 147B overflow structure. The area downstream of the Sub-Basin 147B overflow structure was included in the model, as flows from these areas influence the water level in the conveyance downstream of the overflow structure and may therefore influence the overflows at the structure. The area for Basin 147 is bounded by Meridian Ave N on the east, Phinney Avenue N on the west, N 50th Street on the north, and N 34th Street on the south. Table 1-A summarizes the study sub-basins calibrated along with their respective areas, associated rain gauge, and flow meter locations used for calibration. The basin areas, overflow locations, overflow outfalls and connection to the North Interceptor are shown in Figures 2-1 and 2-2 and described in detail below.

Table 1-A: Study Area Model Basins

Sub-Basin	Sub-Basin Area (acres)	Rain Gauge	Flow Meter
147A	201	RG09	022-187
147B	94	RG09	022-160
147B DS (1)	18.8	RG09	Not Metered

(1) 147B DS represents areas between the overflow structure and the connection to the North Interceptor

Sub-Basins 147A and 147B have separate connection points to the North Interceptor. They share a common CSO outfall to Lake Union and are National Pollutant Discharge Elimination System (NPDES) permitted and monitored as a single discharge location.

Sub-Basin 147A is a 201-acre area that is mostly part of the CSS with the northern-most and western-most portions being partially-separated, meaning that a portion of the stormwater in those areas is conveyed in a separate pipe and does not enter the combined system. The stormwater pipe discharges to Lake Union.

The overflow structure for Sub-Basin147A is a side-cast weir located in City of Seattle MH 022-187. Normal flows are conveyed approximately 50 feet to the connection with the King County North Interceptor. Overflows are conveyed to SPU MH 022-186 where they combine with overflows from Sub-Basin 147B and then continue to an outfall in Lake Union.

Sub-Basin 147B includes a 94 acre catchment of partially-separated sewer area that is tributary to the overflow structure in MH 022-160. From the overflow structure, normal flows (DWF) are conveyed approximately 2500 feet to the west to the connection with the King County North Interceptor. There are additional areas tributary to the conveyance line between the overflow structure and the connection to the North Interceptor. These additional areas are downstream of and therefore not tributary to the 147B overflow structure. Based on an interpretation of available GIS information these lateral connection inputs include an estimated 3.4 acres of impervious and 0.3 acres of RDI tributary area. It was important to include flows from these areas in the model as they result in an increase in the water surface elevation within this downstream portion of the system. The higher tail water condition affects water levels at the overflow weir and effectively increases the overflow volumes.

The overflow structure for Sub-Basin 147B is a side-cast weir located in City of Seattle MH 022-160. Overflows are conveyed to MH 022-186 where they are combined with overflows from Sub-Basin 147A and continue to Outfall 147 in Lake Union.

For each of these CSO sub-basins, flap gates are installed just upstream of the connection with the North Interceptor to prevent flows from the North Interceptor from backing up into the local conveyance toward the overflow weirs.

2 Study Sources

Data collected from a variety of sources were used to develop and calibrate the hydrologic and hydraulic models. Hydrology for the basin was characterized from aerial photography, contour data, evaporation, rainfall, and existing delineations of hydrologic basins (from King County's *Runoff-Transport* model). Hydraulics for the collection system were defined based on as-built drawings, GIS sewer coverages, SCADA, and flow meters. Additionally, portions of the SPU modeling report were used as secondary data sources or to verify inferences and approximations.

2.1 Aerial Photography

Detailed 2012 ortho-rectified aerial photographs procured by King County were overlain in GIS to assess basin land use, which was used in part to characterize basin hydrology. Land uses in the study area include a mix of single family residential, multi-family residential, and commercial areas.

2.2 Contour Data

GIS overlays of contour data were used to qualitatively characterize the slope of ground runoff generating surfaces within the study area. Due to the generally uniform slope of the basin, this was generally only used as an order of magnitude verification of the survey data for the pipes and manholes.

2.3 GIS

Point-based shape files of sewer manholes, and line-based files of sewer pipes, were the primary source of the network data used to parametrize the hydraulic model network. Manhole data includes name, diameter of the pipe intersected, and elevations of the rim and of each inlet and outlet pipe. Pipe data include length, diameter, upstream and downstream invert elevations, and material. Polygon-based shapefiles of KC-WTD CSO basins provided the foundation for the basins used in the hydrologic model.

2.4 Evaporation

A long-term evaporation record was downloaded from the AgWeatherNet Washington State University Puyallup site. This record was averaged by month into an average evaporation year, which was repeated for the duration of the calibration and long-term period of simulation. Both this data source and methodology are commonly used for continuous hydrologic modeling at King County.

2.5 Rainfall

Processed rainfall records from 1978 to 2015 were provided by SPU at 17 rain gauges located within the City of Seattle. These records consist of a continuous one-minute time series of rainfall depth. Rainfall data from SPU Rain Gauge 9 (RG09) were used in hydrologic model calibration. RG09 is located within and at the north end of the study area as shown in Figure 2-1.

2.6 As-built and Design Engineering Drawings

Archived record drawings of sewer pipes and facilities were available from both WTD and SPU. As-Built, design drawings and survey information were used to provide modeling detail for facilities and pipes and to address discrepancies and missing data from GIS shape files. These data sources were especially crucial in accurately modeling the physical parameters of the overflow structures.

2.7 SCADA

WTD maintains a historical record of SCADA (Supervisory Control and Data Acquisition) continuously collected from WTD offsite facilities. SCADA data useful for model calibration includes calculated and measured flows, water levels, gate positions, and operational states of pumps and flow regulating structures. SCADA water level data from the Canal Street weir were available from July, 2005 through December, 2015. These data were used to define a time series for the downstream water level boundary conditions at the 147A and 147B connection points to the North Interceptor. The methodology for transposing these data was slightly different for each connection point as described in Section 3.2.2.2.

Long-term Lake Washington water level data published by the US Army Corp of Engineers were used as a boundary condition at the overflow outfall to Lake Union.

2.8 Flow Monitoring

WTD and SPU perform conveyance system and overflow monitoring with an array of permanent and portable flow meters. These are generally installed in manholes, and log depth and velocity data from which flow rates can be calculated. Data is typically sampled at either 5 or 15 minute intervals. The data are used for the hydrologic and hydraulic model calibration.

Table 2-A below identifies the flow meters and flow data time periods used for the hydrologic and hydraulic model calibration, while Figure 2-2 on the following page shows their locations within the basin. Portions of the flow data used for this project were used in a previous SPU modeling effort. A more detailed description of the flow meter data follows. The meters are ADS FlowShark brand and each installation includes a velocity sensor, a pressure depth sensor and an ultra sonic depth sensor.

Table 2-A: Flow Meter Data Quality and Locations

Flow Meter	Use	Data Quality	Start Date	End Date	Address
022-187	147A Hydrologic / Hydraulic Calib	Good	10/1/2008	4/30/2016	Stone Way N and N 34th Street
022-160	147B Hydrologic / Hydraulic Calib	Fair	10/1/2008	6/30/2016	Woodland Park Ave N and N 34th St
022-186	Hydraulic/Overflow Calib 147A & 147B	Good	Nov. 2010	Present	Stone Way N and N 34th Street

2.8.1 Flow Meter - SPU MH 022-187

The data from flow metering in the invert of the inflow pipe (MP-1) at SPU MH 022-187 were used for hydrologic model calibration in Sub-Basin 147A. These data are considered to be good quality and show a DWF pattern representative of the tributary area that includes single and multi-family residential and commercial areas. The data show a significant response to most rainfall events occurring within the basin, including the fast and slow response runoff components.

Prior to October, 2010 additional sensors installed in invert of the overflow outflow pipe on the downstream (wet weather flow) side of the overflow weir measured the overflows for this basin. Flow turbulence at this location resulted in poor quality data and the overflow monitoring location was moved to the next downstream manhole, at SPU MH 022-186.

2.8.2 Flow Meter - SPU MH 022-160

The data from flow metering in SPU MH 022-160 were used for model calibration in Sub-Basin 147B. These data are considered to be good quality and show a DWF pattern representative of the tributary area that includes single and multi-family residential and commercial areas. As expected with most of the basin being separated or partially separated, the data show a less significant response to rainfall events than in Sub-Basin 147A.

Prior to October, 2010 additional sensors installed in invert of the overflow outflow pipe on the downstream (wet weather flow) side of the overflow weir measured the overflows for this sub-basin. Flow turbulence at this locations resulted in poor quality data and the overflow monitoring location was moved to the next downstream manhole, at SPU MH 022-186.

2.8.3 Flow Meter - SPU MH 022-186

Prior to October, 2010 the overflows for Sub-Basins 147A and 147B were monitored in the overflow structure immediately downstream of the respective overflow weirs in the invert of the overflow outflow pipe. Flow turbulence at those locations resulted in poor quality data. Overflows from Sub-Basins 147A and 147B are conveyed to SPU MH 022-186 and from there to a common outfall in Lake Union.

Beginning in October 2010, overflows from these basins were measured at MH 022-186. Monitoring equipment installed in the inverts of the inflow pipes to MH 022-186 measures flow from the two sub-basins separately and the quality of the flow data at this location is considered more reliable than the previous locations. In August of 2015, the monitoring equipment at this location was replaced with similar but newer generation equipment.

2.8.4 Water Level Meter KC_NI022-185A

Water levels in the North Interceptor were measured in MH KC_NI022-185A where flows from Sub-Basin 147A discharge to the Interceptor. Data from this location, which are available intermittently between September, 2009 and April 2015, were used in the downstream water level boundary condition in the 147A model as described in Section 3.2.2.2.

2.8.5 Flow Meter - SPU MH 022-188

The flap gate upstream of the Sub-Basin 147A connection point to the North Interceptor is located in SPU MH structure 022-188. Flow monitoring in this structure on both the upstream and downstream sides of the flap gate provided data used to develop the operational curve used to represent the flap gate in the model. Development of the operational curve using data from this meter location is described in Section 3.2.3

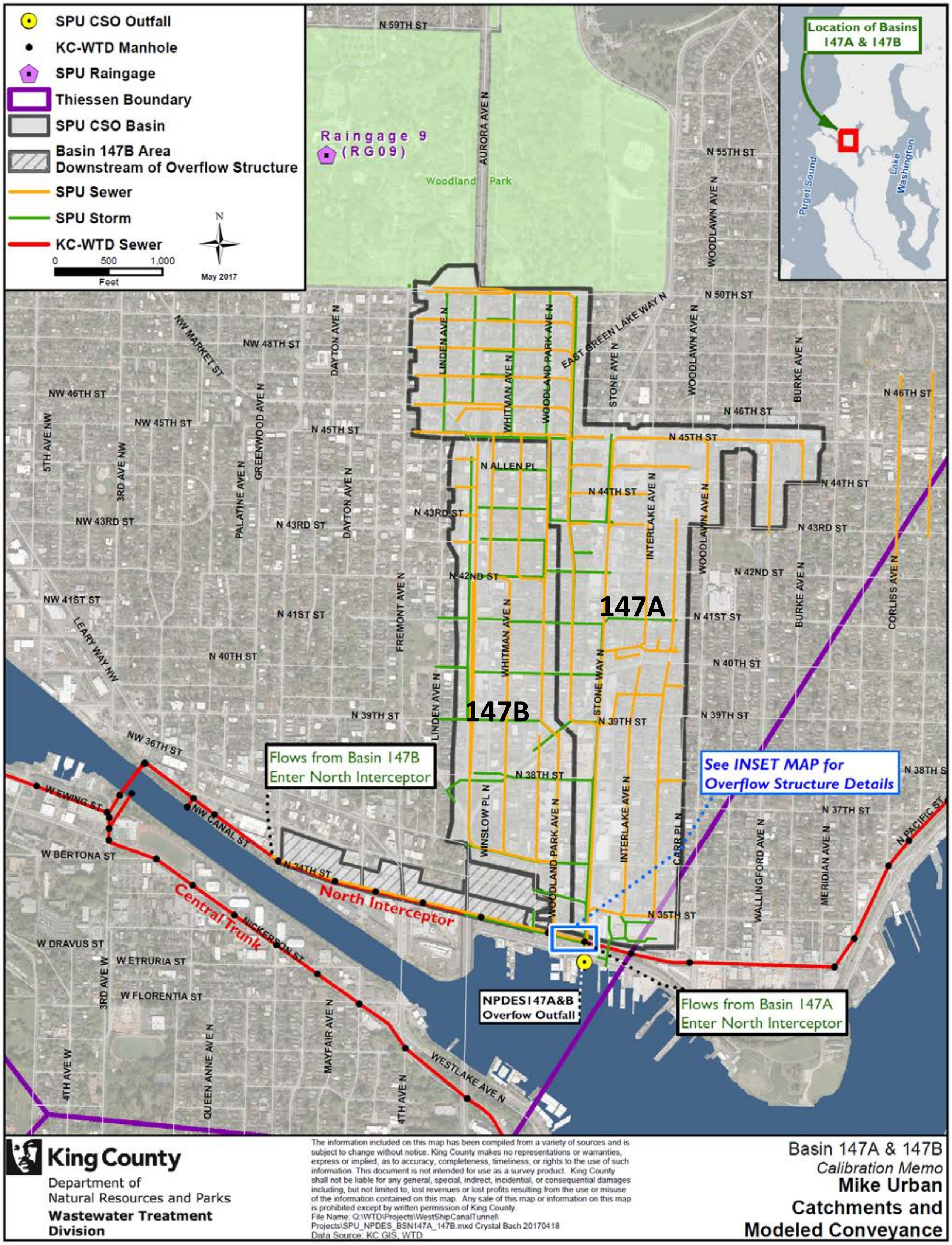


Figure 2-1: NPDES Basins 147A and 147B Areas

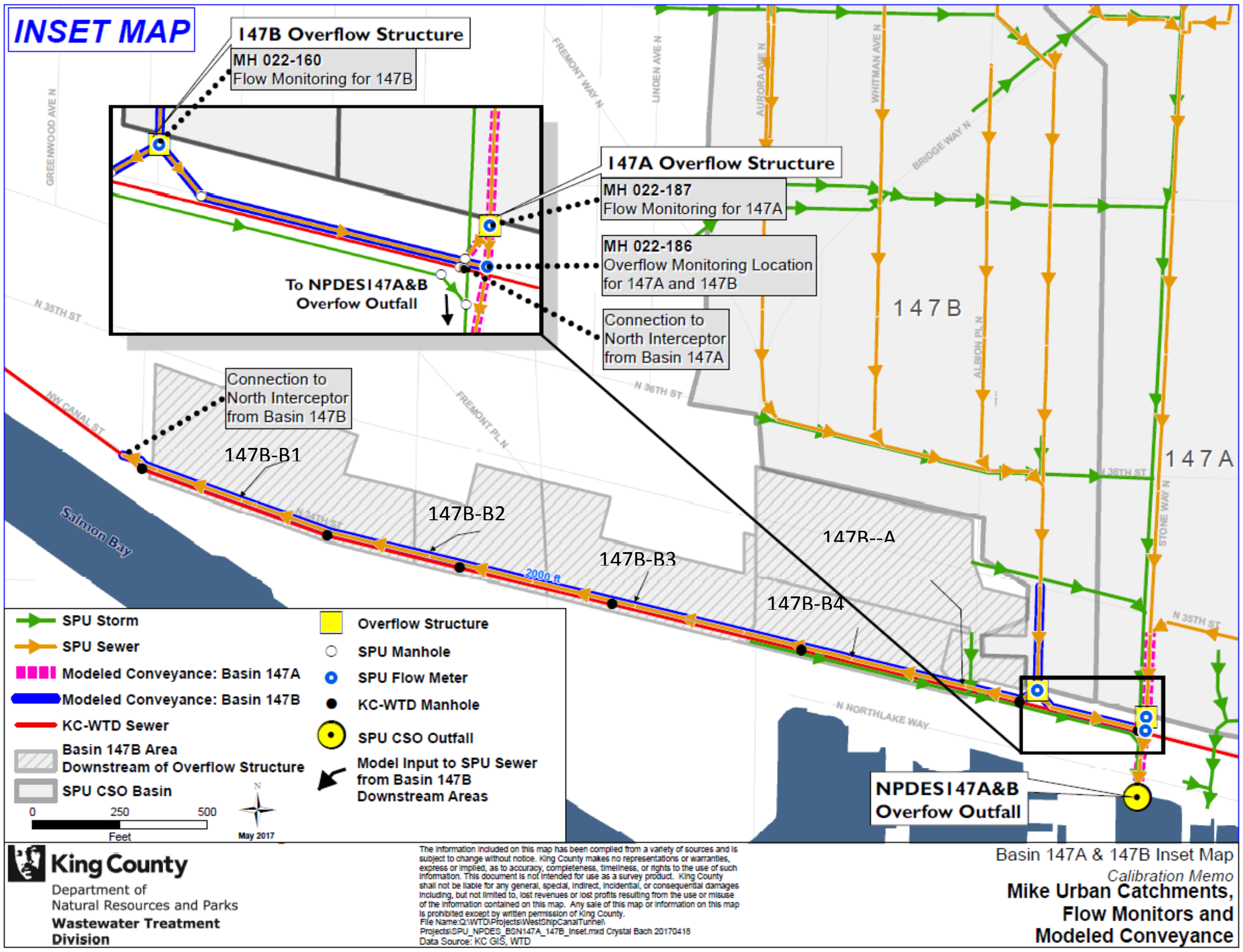


Figure 2-2: Basin 147 Overflow Structures, Outfall and Connections to North Interceptor

3 Model Development

Model calibration is the process of iteratively adjusting model parameter values until the results of the model most closely approximate real world observations. In this effort, model calibration involved adjustment of both hydrologic and hydraulic model parameters as described in the following subsections. The final model calibration results are presented in Section 4.

The Danish Hydraulic Institute (DHI) MIKE URBAN model was selected to perform hydrologic and hydraulic modeling of the Ballard CSS. The model consists of a hydrologic component and hydraulic component.

Model development consisted first of compiling the different elements of the model (basins, manholes, pipes, and control structures, which, in the case of this particular basin, included the two overflow weirs), connecting them, and assigning them properties consistent with their physical attributes. Second, rainfall, evapotranspiration, dry weather flows (DWF), and Salmon Bay and Ballard RS trunk water levels were used to define the forcing data and boundary conditions for the model. Third, the head losses at nodes were adjusted to better match observed data and to ensure model stability, as the default parameters in MIKE URBAN tend to compute losses that are higher than monitoring data would suggest are realistic.

3.1 Hydrologic Model Development and Calibration

The hydrologic component (MOUSE RDII [Rainfall Dependent Infiltration and Inflow] and MOUSE RO Runoff Computation) characterizes the basin response to rainfall in terms of a hydrograph at a tributary location. The hydrograph consists of both storm runoff and infiltration (RDI). Surface runoff routing is calculated using the Kinematic Wave model (Model B), and accounts for runoff produced from impervious areas of the basin. It is also referred to as the fast response component (FRC). The RDI module accounts for overland flow, interflow, and groundwater processes related to pervious areas of the basin. It is also referred to as the slow response component (SRC). RDII considers evapotranspiration, soil moisture, and groundwater table depth in its calculations.

The FRC requires length and slope parameters for each basin. These were constants in the model set to 5% and 328 ft, respectively. A 5% slope is representative of basin topography and the runoff generating surfaces in both Sub-Basins 147A and 147B. The parameters adjusted through the calibration process for each sub-basin are presented in Table 3-A and the rest of the hydrologic parameters are set to MOUSE default values.

Table 3-A: Parameters Calibrated for the Hydrologic Model

Parameter Name	Model ID	Description	Units
Impervious Area – Flat	B_A_IFLAT	Fraction of basin area that is impervious and connected to the CSS	%
Impervious Manning Number - Flat	B_M_IFLAT	Defines the roughness of the impervious basin area, used in the hydraulic routing of the runoff (Manning's equation)	N/A
RDII Area	RDII_AREA	Fraction of basin that contributes RDII to the CSS	%

Surface Storage	UMAX	Defines the maximal water contents in surface storage	In
Root Zone Storage	LMAX	Defines the maximal water contents in root zone storage	In
Overland Coefficient	CQOF	Determines the extent to which excess rainfall (after surface storage is retained) runs off as overland flow	N/A
Time Constant	CK	Determines how fast the flow responds to rainfall. Also has some effect on the routing of interflow.	Hr
Time Constant Interflow	CKIF	Together with Umax determines the amount of interflow	Hr
Time Constant Baseflow	CKBF	Determines the hydrograph recession during dry periods	Hr

Initial hydrologic calibration for each basin was performed through an automated process using Model-Independent Parameter Estimation and Uncertainty Analysis (PEST) software. Following the PEST calibration, a manual adjustment of the parameters was carried out to improve the match between simulation results and field data.

Hydrologic calibration required identification of continuous time periods for which reliable flow and rainfall data were available. The period used for hydrologic calibration for Sub-Basins 147A and 147B was November, 2008 through January, 2010. Good quality flow data for Sub-Basin 147A in 2014 and 2015 allowed for verification of the model calibration. The quality of the flow data collected in Sub-Basin 147B during 2014 and 2015 was poor and therefore could not be used for verification of the model calibration.

Specific storm events suitable for hydrologic model calibration were identified through a detailed review of available flow data. Ten flow events of varying magnitude and duration were identified for each of the study sub-basins. As standard practice with hydrologic modeling, a spin-up period of at least two wet seasons prior to the first calibration event was simulated.

The boundary condition data for the hydrologic model are rainfall and evapotranspiration. These data were described in Section 2.

3.1.1 PEST Weighting

PEST has the ability to weight individual observations during the calibration. PEST will dedicate more computational effort to matching observations with weights above 1, and less effort for weights below 1. Observations with zero weight are effectively ignored.

The selection of an appropriate weighting strategy is dependent largely upon the nature of the calibration and PEST's performance without a weighting scheme. For these model basins, the calibration performed adequately without the use of a variable weighting scheme. All valid values were assigned a weight of 1, and missing or clearly inaccurate data were assigned a weight of 0 to discount their contribution from the calibration error measurement and decision process. One segment of the flow time series was assigned a weight of 0 for both Sub-Basin 147A and Sub-Basin 147B. The period from January 4th, 2009 at 13:00 to January 5th, 2009 at 21:00 includes snowfall, which results in a mismatch in the timing of the response flow and was, therefore, unfit for calibration purposes.

3.1.2 Manual calibration

After the PEST calibration, an analysis of the resulting RDII flow components showed which of those were under- or over-estimated. Adjustment of the parameters based on this information improved the model fit to data for most basins. In general, manual calibration was used to adjust the hydrologic parameter *B_A_Iflat* and was used to attain a more desirable fit for the peak flow values in cases where the model consistently under- or over-estimated the peak flow values.

3.1.3 Hydrologic Model Calibration Evaluation Metrics

The PEST and manual calibration process involved iteratively running a simulation, evaluating simulation results and adjusting model parameter values to improve the results. The effectiveness of model parameter adjustments during the calibration process were evaluated based on goodness-of-fit of the simulated results versus measured data through visual observation of plotted data using the standardized Bias, Nash-Sutcliffe model efficiency coefficient, the Root-Mean-Square Error (RMSE), the Relative Peak Flow Difference and Relative Total Volume Difference. The following subsections present more detail on each of these calibration metrics.

3.1.3.1 Bias

Bias indicates a general shift of the models, and ranges from positive to negative infinity, where 0.0 indicates a perfect fit. Positive and negative bias indicates model overestimation and underestimation, respectively. Bias is expressed as:

$$B_s = \frac{\sum_{i=1}^{m}\left(Q_i^{mdl} - Q_i^{obs}\right)}{m * \overline{Q^{obs}}}$$

where m is the number of values in the calibration event, Q_i^{mdl} is a model value, Q_i^{obs} is an observed value, and $\overline{Q^{obs}}$ is the average observed value.

3.1.3.2 Nash-Sutcliffe Coefficient

The Nash coefficient is one less the ratio of the sum of the squared differences between modeled and observed values and the sum of the squared differences between the observed and mean observed values. Nash values can range from 1.0 to negative infinity, where 1.0 indicates a perfect fit. In practice, the Nash coefficient served as the primary goodness-of-fit indicator. Values of 0.9 and greater indicated excellent agreement between the observations and the model. The Nash coefficient is expressed as:

$$Nash = 1 - \frac{\sum_{i=1}^{m}(Q_i^{mdl} - Q_i^{obs})^2}{\sum_{i=1}^{m}(Q_i^{obs} - \overline{Q}_i^{obs})^2}$$

where m is the number of values in the calibration event, Q_i^{mdl} is a model value, Q_i^{obs} is an observed value, and $\overline{Q_i}^{obs}$ is the average observed value in the event.

3.1.3.3 Root-Mean-Square Error

RMSE is the square root of the average squared difference between the observed and model values. RMSE values can range from 0.0 to positive infinity, where 0.0 indicates a perfect fit. RMSE is expressed as:

$$RMSE = \sqrt{\frac{1}{m}\sum_{i=1}^{m}(Q_i^{mdl} - Q_i^{obs})^2}$$

where m is the number of values in the calibration event, Q_i^{mdl} is a model value, and Q_i^{obs} is an observed value.

3.1.3.4 Relative Peak Flow Difference and Relative Total Volume Difference

Two additional metrics further quantified the goodness-of-fit. These compared the relative difference of model peak from observed peak, and total model volume from total observed volume. Values of 0.0 for both peak flow and total volume indicate perfect agreement. Positive or negative values indicate model overestimation or underestimation, respectively. The relative peak flow and relative total volume are expressed as:

$$Q_{pk\_diff} = \frac{Q_{\max}^{mdl} - Q_{\max}^{obs}}{Q_{\max}^{obs}} \quad ; \quad V_{diff} = \frac{V_{tot}^{mdl} - V_{tot}^{obs}}{V_{tot}^{obs}}$$

where $Q_{\max}^{mdl}$ is the maximum model value and $Q_{\max}^{obs}$ is the maximum observed value, V_{tot}^{mdl} is the total model volume, and V_{tot}^{obs} is the total observed volume.

3.2 Hydraulic Model Development and Calibration

The hydraulic component (MOUSE HD Pipe Flow Computation) further characterizes the hydrologic response to rainfall in terms of flow and water levels within the system. MOUSE HD Pipe Flow uses a Dynamic Wave model to simulate routing of flow within a network of nodes and links. Nodes represent manholes or outlets; links represent pipes, storage tanks and control devices such as pumps, valves, weirs and regulated gates. The hydraulic network represented in the Basin 147 models is highlighted in Figure 2-1 with pink (dashed) and blue, respectively. The modeled conveyance is minimalized as the calibration point for each of these basins is at the overflow structure. The primary focus and purpose of hydraulic model calibration was to simulate the effects of the conveyance downstream of the overflow weir including the backwater conditions created by the water levels in the North Interceptor and allow for accurately simulating overflow occurrences and volumes for these model basins.

Both basin models include an overflow weir represented as a rectangular orifice and a flap gate upstream of the connection to the North Interceptor to prevent reverse flow from the North Interceptor into the connecting local line. However, the specific as-built details and, therefore, the hydraulic function of these structures, are unique to the individual basins. The data used to represent these structures in the model as well as adjustments made during the calibration process are discussed in the following subsections.

3.2.1 Losses in Nodes

Head losses through the conveyance system features affect the water surface elevations and therefore flow over the weirs. Head losses in nodes due to manholes and junctions are computed in MIKE Urban. The standard calculation usually overestimates the losses, and thus custom losses were set for the nodes in the models. The custom loss estimates follow the Weighted Inlet Energy method (also referred to as Mean Energy Approach), with a coefficient type of total head loss (Total HLC) applied at the outlet of the node. Five standard cases were used in this model. The loss coefficients used are listed in Table 3-B.

Table 3-B: Head Loss Coefficients

Type of Node	Loss Coefficient
45 Bend	.1
Junction_sml	1 *(1)*
Channelized	.05
Junction	No head loss

Notes: (1) Used for representing head losses in the hydraulic calibration

3.2.2 Hydraulic Model Boundary Conditions

The boundary conditions for the hydraulic model include dry weather flow (DWF) hydrographs for each sub-basin, downstream water levels in the North Interceptor, and water levels in Lake Union for the overflow outfalls.

3.2.2.1 Dry Weather Flows

The DWF is the combination of groundwater infiltration and sanitary flow regularly entering the collection system regardless of precipitation. Meter data during dry weather periods were used to create a set of dry weather diurnal curves for all basins. Three diurnal curves were created for each basin to reflect average observed flows during weekdays, Saturdays, and Sundays. The 5-minute flow meter data was averaged into hourly flow rates to get the 3 DWF patterns for each basin. Flow meter data from the dry weather period from May through August 2009 was used to derive representative DWF averages.

3.2.2.2 Downstream Water Levels

The hydraulic models for each of the CSO basins include two outlets; the overflow outfall to Lake Union and the connection to the North Interceptor. At each of these outlets, a water level time series was used as a downstream boundary condition. For each overflow outfall, the Lake Union water level was represented using a daily average Lake Washington water level per the US Army Corp of Engineers data.

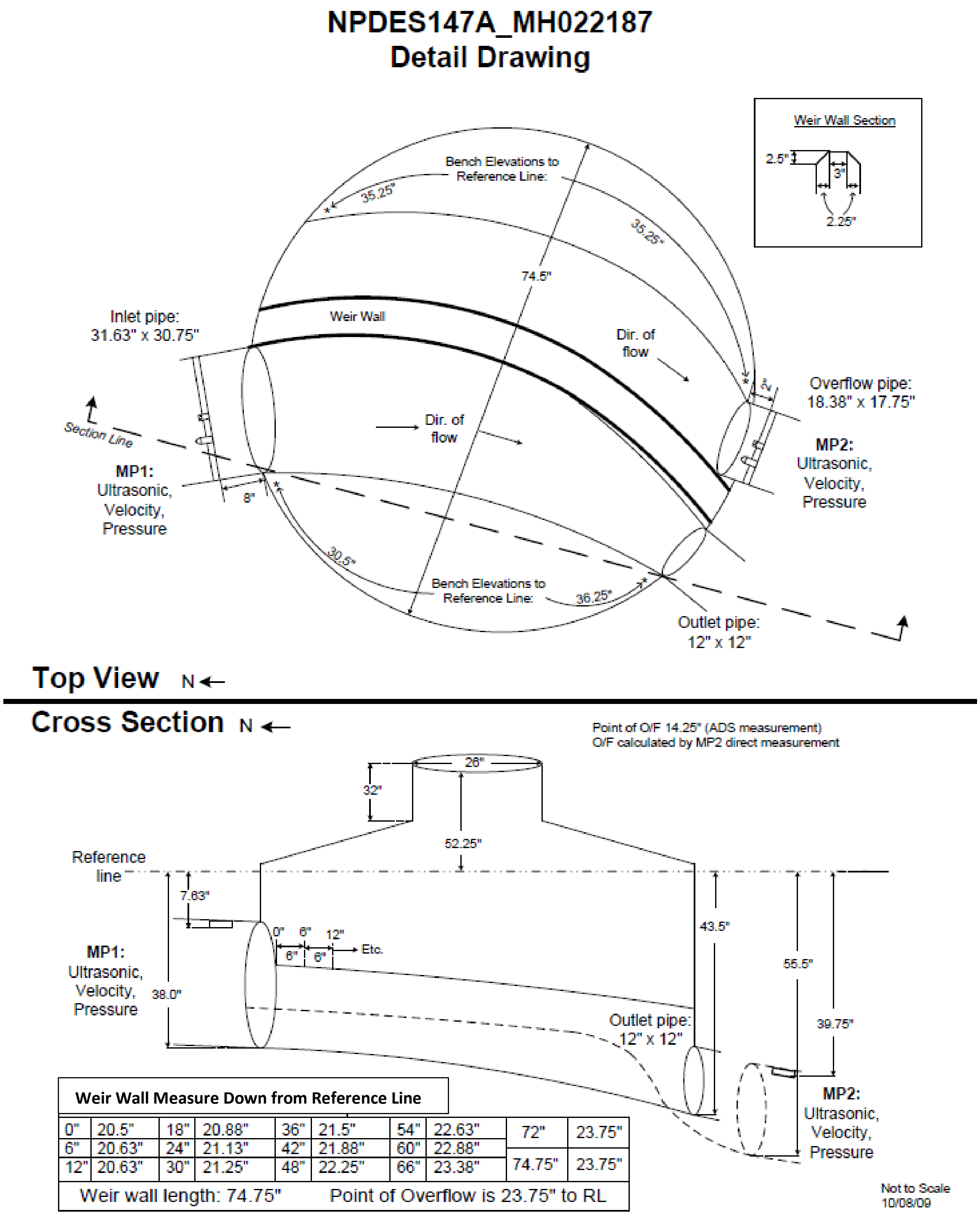
At the outlet of the models into the North Interceptor, water level boundary condition data are a combination of measured data near the point of connection, SCADA data from the Canal Street Weir and output from UNSTDY model long term simulations.

Water level data collected in the North Interceptor at the Sub-Basin 147A connection point, MH KC_NI022-185A, were available intermittently between September, 2009 and April, 2015. A scatter plot of these data versus SCADA water level data from the Canal Street weir was used to characterize the relationship between the two measured data sets. That mathematical relationship was then used to transpose the Canal Street weir SCADA data to this Sub-Basin 147A connection point and produce a longer-term, composite data set which was used as the downstream boundary condition.

For the Sub-Basin 147B model North Interceptor downstream boundary condition, the Canal Street weir SCADA water level data were transposed to the Sub-Basin 147B - North Interceptor connection point based on the slope of the North Interceptor invert between the two locations.

3.2.3 Sub-Basin 147A Control Structures - Overflow Weir and Flap Gate

The Sub-Basin 147A overflow weir which is located in MH 022-187 is a curved, side-cast weir as represented in Figure 3-1. The weir was modified with a retrofit in October, 2010 effectively raising the crest 0.26 feet from elevation 122.68 feet to 122.94 feet (King County Datum, subtract 96.41 feet to convert to NAVD88). With the post-retrofit scenario, overflows at the weir occur when depths on the upstream side of the weir reach approximately 2.02 feet. The weir was represented in the model as a rectangular orifice with parameter values as presented in Table 3-C.



Weir Wall Measure Down from Reference Line

0"	20.5"	18"	20.88"	36"	21.5"	54"	22.63"	72"	23.75"
6"	20.63"	24"	21.13"	42"	21.88"	60"	22.88"		
12"	20.63"	30"	21.25"	48"	22.25"	66"	23.38"	74.75"	23.75"

Weir wall length: 74.75" Point of Overflow is 23.75" to RL

Figure 3-1: NPDES 147A Overflow Weir Structure

Table 3-C: MIKE URBAN Orifice Parameters for NPDES 147A Overflow Weir

Weir Type:	**Side-Cast Weir**	
Oper. Mode:	No Control	
Weir Crest Elev. (Pre/Post Retrofit):	122.68/122.94	Ft
Flap:	FALSE	
Discharge coefficient:	1	
Height from weir crest to vault ceiling:	9.3	Ft
Width of weir:	5.6	Ft

The flap gate located upstream of the Sub-Basin 147A conveyance connection to the North Interceptor is intended to prevent reverse flow from the North Interceptor to the local system. When water levels in the North Interceptor are high relative to water levels on the upstream side of the flap gate, flow through the flap gate is restricted. The flap gate, which is located in SPU MH 022-188 is not completely effective and allows reverse flows when the water level in the North Interceptor is higher than the water level in the SPU local line upstream of the flap gate. Flow data collected on the upstream and downstream sides of the 147A flap gate confirm the flap gate is "leaky". The flow data, which were collected and processed in support of a previous SPU modeling effort performed by Brown and Caldwell, were used in that previous effort to develop a relationship between head differential head across the flap gate and flow through the flap gate. The resulting curve, which is presented in Figure 3-2, was applied in this current modeling effort to simulate the flap gate as a regulation pipe. The reverse ("leaky") flow component was simulated using a reverse flow orifice.

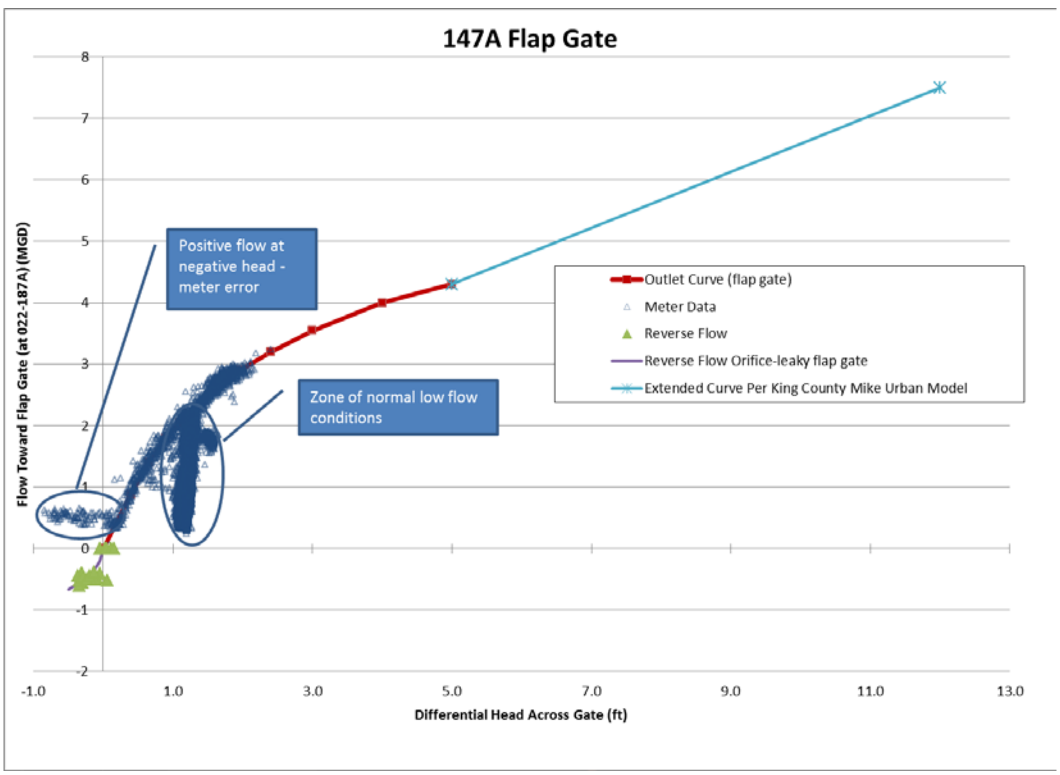


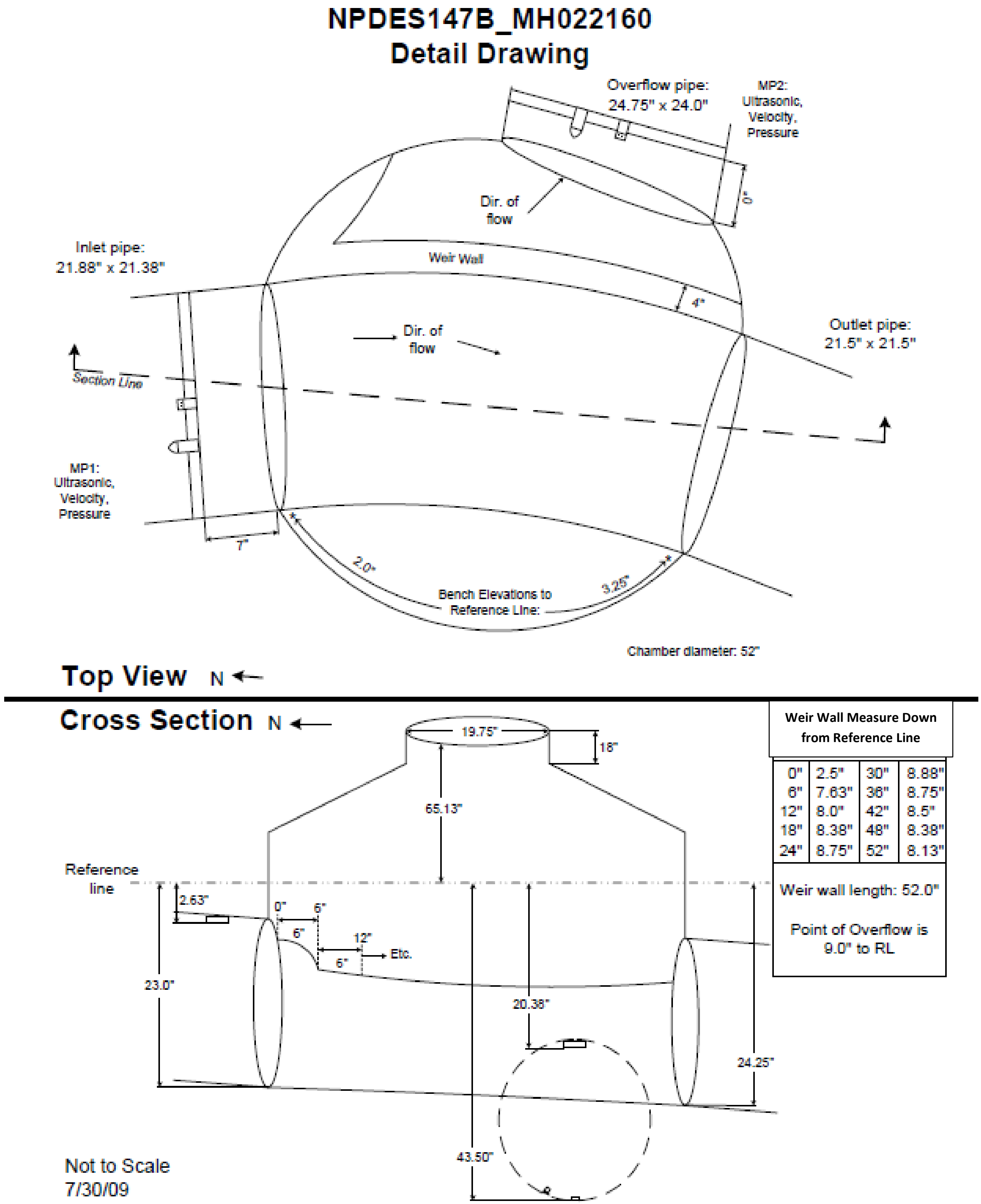
Figure 3-2: Sub-Basin 147A Differential Head – Discharge Curve for Flap Gate

High water levels in the North Interceptor results in a backwater in the conveyance between the overflow structure and the flap gate. As previously described and as indicated in the differential head curve in Figure 3-2, a reverse head differential across the flap gate may also result in reverse flows from the North Interceptor towards the overflow weir. In any case, when water levels are high in the North Interceptor, flow through the flap gate is restricted, and flows from Sub-Basin 147A back up filling the limited storage in the conveyance pipe downstream of the weir, raising the water level at the overflow weir resulting in overflows. The reverse flow through the "leaky" flap gate also contributes to weir overflow volumes.

In addition, flow and water level data collected on the overflow side of the weir indicate the occurrence of a submerged weir, resulting from the hydraulic restrictions in the conveyance downstream (on the overflow side) of the weir. In order to simulate this submerged weir scenario, two dummy nodes were added in the model to provide a mechanism for simulating head losses through that portion of the system.

3.2.4 Sub-Basin 147B Control Structures - Overflow Weir and Flap Gate

The Sub-Basin 147B overflow weir, which is located in MH 022-160, is a curved, side-cast weir as represented in Figure 3-6. The weir was modified with a retrofit in October, 2010, effectively raising the crest 0.42 feet from elevation 123.85 feet to 124.27 feet. With the post-retrofit scenario, overflows at the weir occur when depths on the upstream side of the weir reach approximately 1.68 feet. The weir was represented in the model as a rectangular orifice with parameter values as presented in Table 3-D.



Weir Wall Measure Down from Reference Line

0"	2.5"	30"	8.88"
6"	7.63"	36"	8.75"
12"	8.0"	42"	8.5"
18"	8.38"	48"	8.38"
24"	8.75"	52"	8.13"

Figure 3-3: Drawing of NPDES 147B Overflow Weir Structure

Table 3-D: MIKE URBAN Orifice Parameters for NPDES 147B Overflow Weir

Weir Type:	Side-Cast Weir	
Oper. Mode:	No Control	
Weir Crest Elev. (Pre/Post Retrofit):	123.85/124.27	ft
Flap:	FALSE	
Discharge coefficient:	1	
Height from weir crest to vault ceiling:	6.17	ft
Width of weir:	4.0	ft

Overflows for Sub-Basin 147B are fewer and lower in volume than those for Sub-Basin 147A. The overflow system hydraulics are less complicated with no apparent “leaky” flap gate and no apparent submerged weir conditions. However, basin flow contributions to the normal (DWF) conveyance between the overflow weir and the connection to the North Interceptor can affect (increase) the water level at the overflow weir and, therefore, increase the overflow volumes.

Field data collected in support of the previous modeling effort indicated the presence of approximately six inches of sediment in the conveyance pipe between the overflow structure and the connection to the North Interceptor. However, field data collected during the current modeling effort indicated little or no sediment in this conveyance line.

The field visit on May 31, 2017 included an inspection of all manholes along the 24-inch SPU local line beginning at the overflow structure, MH 022-160 down to MH 022-177, the downstream-most manhole upstream of the connection to the North Interceptor. Little or no sediment was observed in the pipe. The inspection involved using a survey rod to prod the flow channel through each structure (where structure depth permitted) and estimate sediment depth. Three of the manholes were too deep (approximately 30 to 40 feet deep) to allow for prodding. For those manholes, visual observation of the flow stream indicated a possible sediment depth in the manhole channel of 2 to 4 inches.

Table 3-E below identifies the estimated sediment depth, the method of estimation and the level of confidence at each of the manholes.

Table 3-E: SPU Local Line Inspection - NPDES 147B Downstream of Overflow Weir (DWF Side)

Manhole ID	Sediment Depth (in)	Estimation Method	Confidence
MH 022-161	1”	sediment prod	moderate
MH 022-162	4”	sediment prod	moderate
MH 022-168	0”	sediment prod	high
MH 022-169	could not open	N/A	N/A
MH 022-170	2”	visual from rim	low
MH 022-171	4”	visual from rim	low
MH 022-172	4”	visual from rim	low
MH 022-173	0”	sediment prod	high
MH 022-174	0”	sediment prod	high
MH 022-416	1”	sediment prod	high
MH 022-176	4”	sediment prod	moderate
MH 022-177	2”	sediment prod	moderate

The conveyance profile in Figure 3-4 below is based on available GIS data and shows that the slope of this pipe is consistent. As such, a somewhat consistent sediment depth in this line would be expected. Local anomalies in manhole channel geometry, slope and roughness could explain these variations in sediment depths within the manhole channels. Inflow through lateral connections and I/I could introduce sediment in some of the manholes and not others. Also, the presence of sediment in the manhole channel does not mean there is sediment in the conveyance pipe.

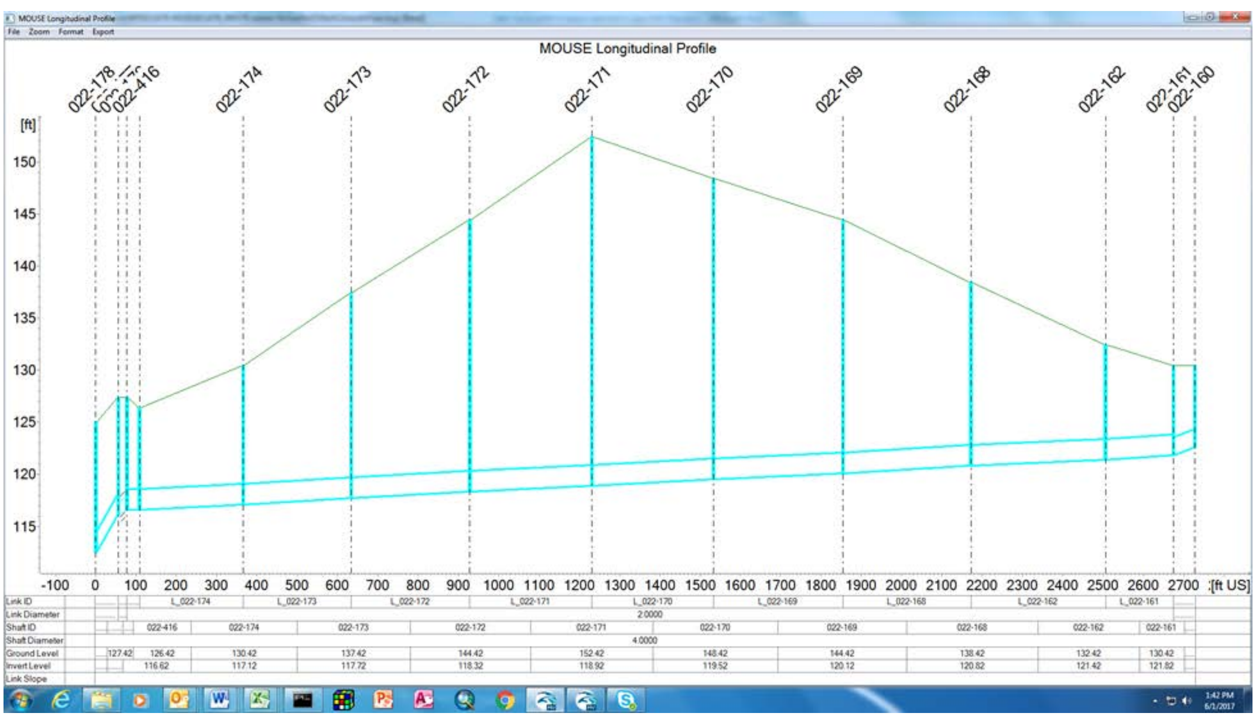


Figure 3-4: NPDES 147B Conveyance Profile from Overflow Structure to North Interceptor Connection

During the field inspection, no inflow from lateral lines was observed in these manholes. It should be noted that the weather was cloudy but it is unlikely that there had been rainfall within the prior 15 hours. With the exception of the a brief but high intensity rain shower approximately 15 hours prior to the inspection, no rainfall had occurred in the prior 6 days.

Based on these field observations, model runs performed to calibrate the hydraulics associated with the overflow structure, flap gate at the connection to the North Interceptor, and the conveyance between these two structures assumed no sediment in the line.

The flap gate was simulated as a circular orifice. The diameter of the orifice (flap gate) as represented in the model was derived through the iterative calibration runs and comparison of simulated to measured overflows. The final calibrated orifice diameter is 1.7 feet.

4 Model Calibration Results

The primary purpose and focus of hydrologic and hydraulic model calibration was to provide for accurately simulating overflow occurrences and volumes for these model basins across a range of rainfall event intensities and magnitudes. Simulated and measured overflow results presented herein include only the post-retrofit (post weir-raise) period. The reliability of the measured overflow data is higher after October 2010 when the overflow measurement location was moved to MH 022-186.

4.1 Hydrologic Calibration Results

Calibrated parameter values for the study basins are presented in Table 4-A and the respective catchments are identified in Figures 2-1 and 2-2. The hydrologic calibration event windows are presented in Table 4-B. Evaluation statistics for Sub-Basin 147A and 147B calibrations are presented in Tables 4-C and 4D, respectively. Graphs showing calibrated model output, measured flow, and precipitation for Sub-Basin 147A and 147B are presented in Figures 4-1 and 4-2, respectively.

Table 4-A: Calibrated Parameter Values for NPDES 147A and NPDES 147B

				147B Areas Tributary to Conveyance Between Overflow Structure and Connection to North Interceptor (1)				
Catchment ID	**147A**	**147B**		**147B-A**	**147B-B1**	**147B-B2**	**147B-B3**	**147B-B4**
RDII_AREA	48%	48%		4.5%	0.9%	0.0%	0.0%	0.4%
B_A_IFlat	22.5%	9.3%		50.0%	0.0%	0.0%	11.7%	10.5%
B_M_IFlat	0.1	0.0675		0.0675	0.0675	0.0675	0.0675	0.0675
UMAX	0.3 in	1.28 in		1.28 in	1.28 in	1.28 in	1.28 in	1.28 in
LMAX	8.36 in	7.2 in		7.2 in	7.2 in	7.2 in	7.2 in	7.2 in
CQOF	0.05	0.23		0.23	0.23	0.23	0.23	0.23
CK	28 hr	2.25 hr		2.25 hr	2.25 hr	2.25 hr	2.25 hr	2.25 hr
CKIF	200 hr	175 hr		175 hr	175 hr	175 hr	175 hr	175 hr
CKBF	964 hr	631 hr		631 hr	631 hr	631 hr	631 hr	631 hr
I_U	0.39 in	0.39 in		0.39 in	0.39 in	0.39 in	0.39 in	0.39 in
I_L	4.43 in	4.43 in		4.43 in	4.43 in	4.43 in	4.43 in	4.43 in
GW_CAREA	1	1		1	1	1	1	1

(1) Areas not calibrated: Impervious and RDII area % estimated from GIS, RDI parameter values assigned from 147B calibration.

Table 4-B: Hydrologic Calibration Event Windows for NPDES 147A and NPDES 147B

Event	147A	147B
1	11/1/2008 - 11/10/2008	11/1/2008 - 11/10/2008
2	12/29/2008 - 1/10/2009	12/29/2008 - 1/10/2009
3	4/1/2009 - 4/8/2009	4/1/2009 - 4/8/2009
4	5/4/2009 - 5/10/2009	5/4/2009 - 5/10/2009
5	9/3/2009 - 9/9/2009	9/3/2009 - 9/9/2009
6	11/6/2009 - 11/8/2009	10/16/2009 - 10/26/2009
7	11/13/2009 - 11/29/2009	11/13/2009 - 11/29/2009
8	12/17/2009 - 12/23/2009	12/17/2009 - 12/23/2009
9	1/10/2010 - 1/17/2010	1/10/2010 - 1/17/2010
10	1/23/2010 - 1/26/2010	1/23/2010 - 1/26/2010

Table 4-C: Goodness-of-Fit Metrics for Sub-Basin 147A Hydrologic Calibration

NPDES 147A

Event	Bias	RMSE	Nash	Qpk Error	Vtot Error
01	0.35	0.74	0.60	125.87 %	43.34 %
02	0.07	0.51	0.76	-7.04 %	-11.13 %
03	0.14	0.59	0.70	58.40%	9.97 %
04	0.26	0.41	0.82	-3.44 %	22.61 %
05	0.28	0.35	0.81	14.26 %	29.84 %
06	0.13	0.56	0.78	32.85 %	10.70 %
07	0.08	0.31	0.93	12.84 %	-1.17 %
08	0.30	0.35	0.70	4.62 %	13.99 %
09	-0.07	0.33	0.92	2.33 %	-7.99 %
10	0.32	0.27	0.73	-10.29 %	27.41 %
Average:	**0.19**	**0.44**	**0.77**	**23.04 %**	**13.58 %**

Table 4-D: Goodness-of-Fit Metrics for Sub-Basin 147B Hydrologic Calibration

NPDES 147B

Event	Bias	RMSE	Nash	Qpk Error	Vtot Error
01	0.36	0.21	0.29	105.92 %	33.53 %
02	0.04	0.17	0.78	-44.02 %	-14.57 %
03	0.34	0.18	0.71	-3.21 %	30.06 %
04	0.29	0.13	0.67	0.66 %	18.58 %
05	-0.04	0.10	0.77	14.77 %	-4.31 %
06	0.00	0.14	0.74	-45.62 %	-4.91 %
07	0.15	0.13	0.84	-26.09 %	6.46 %
08	0.23	0.10	0.58	19.21 %	13.85 %
09	0.21	0.16	0.76	-25.11 %	24.99 %
10	0.31	0.10	0.47	-15.34 %	24.51 %
Average:	**0.19**	**0.14**	**0.66**	**-1.88 %**	**12.82 %**

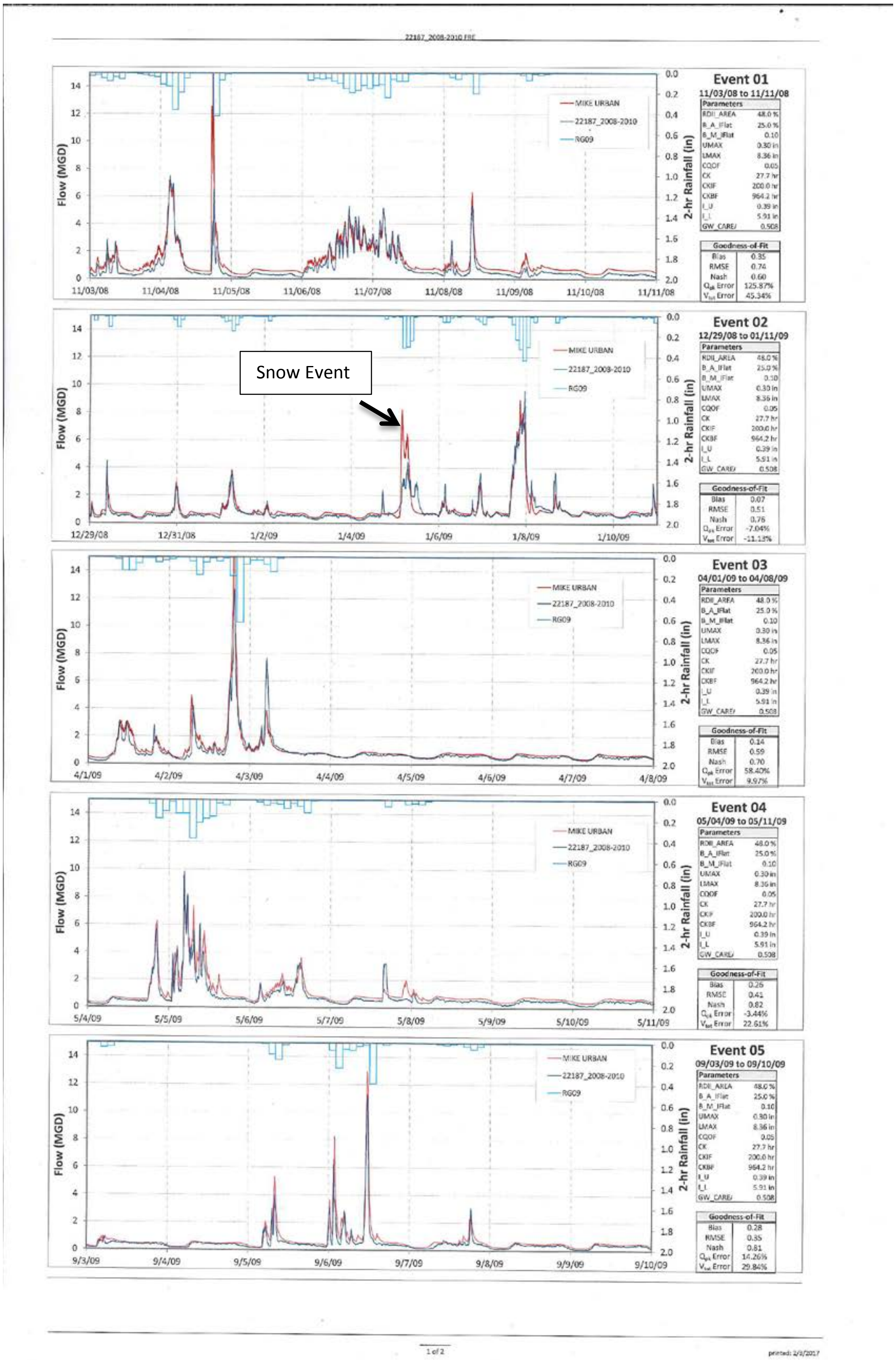


Figure 4-1: Sub-Basin 147A Hydrologic Calibration Graphs

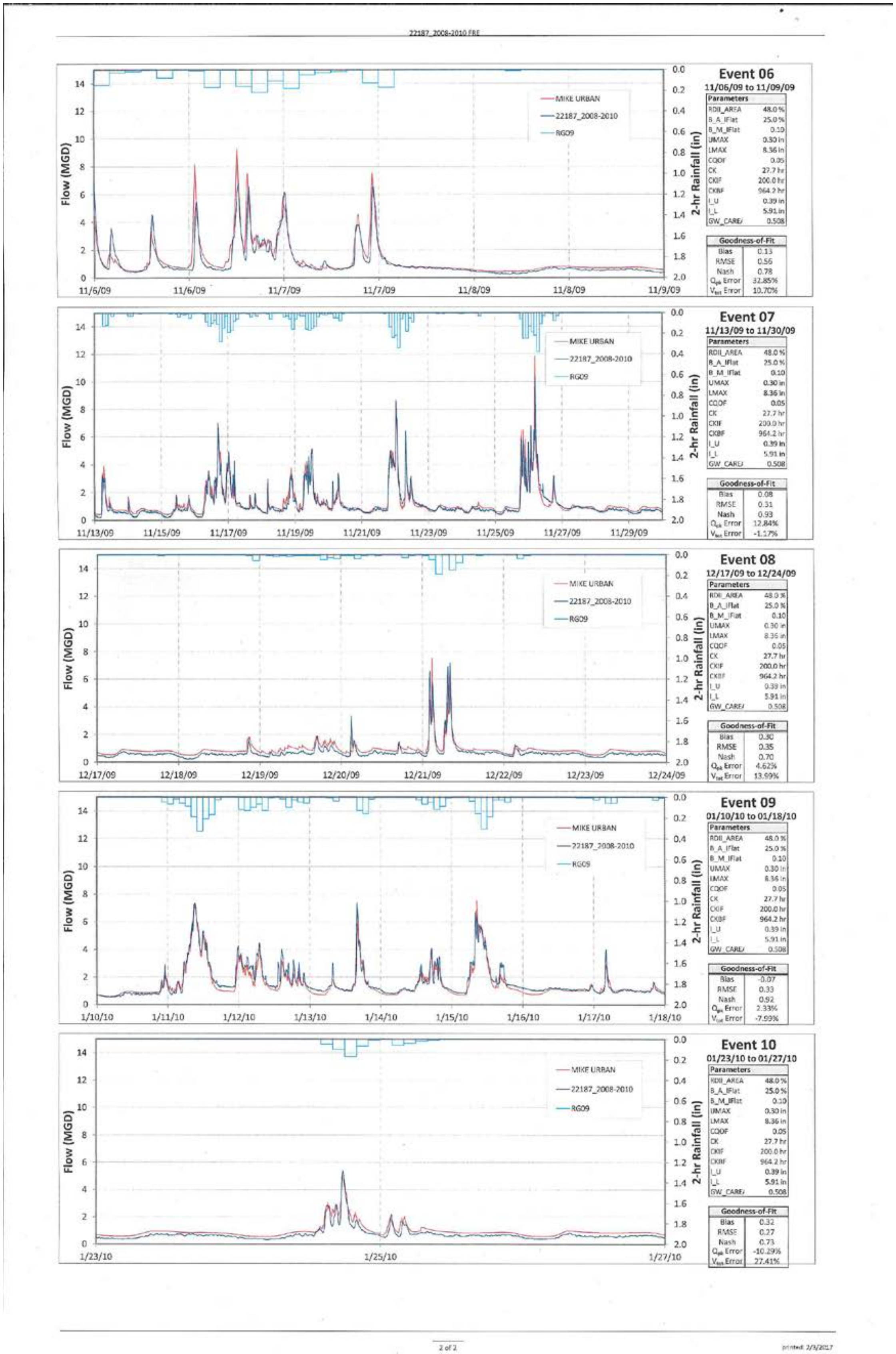


Figure 4-1 (Continued): Sub-Basin 147A Hydrologic Calibration Graphs

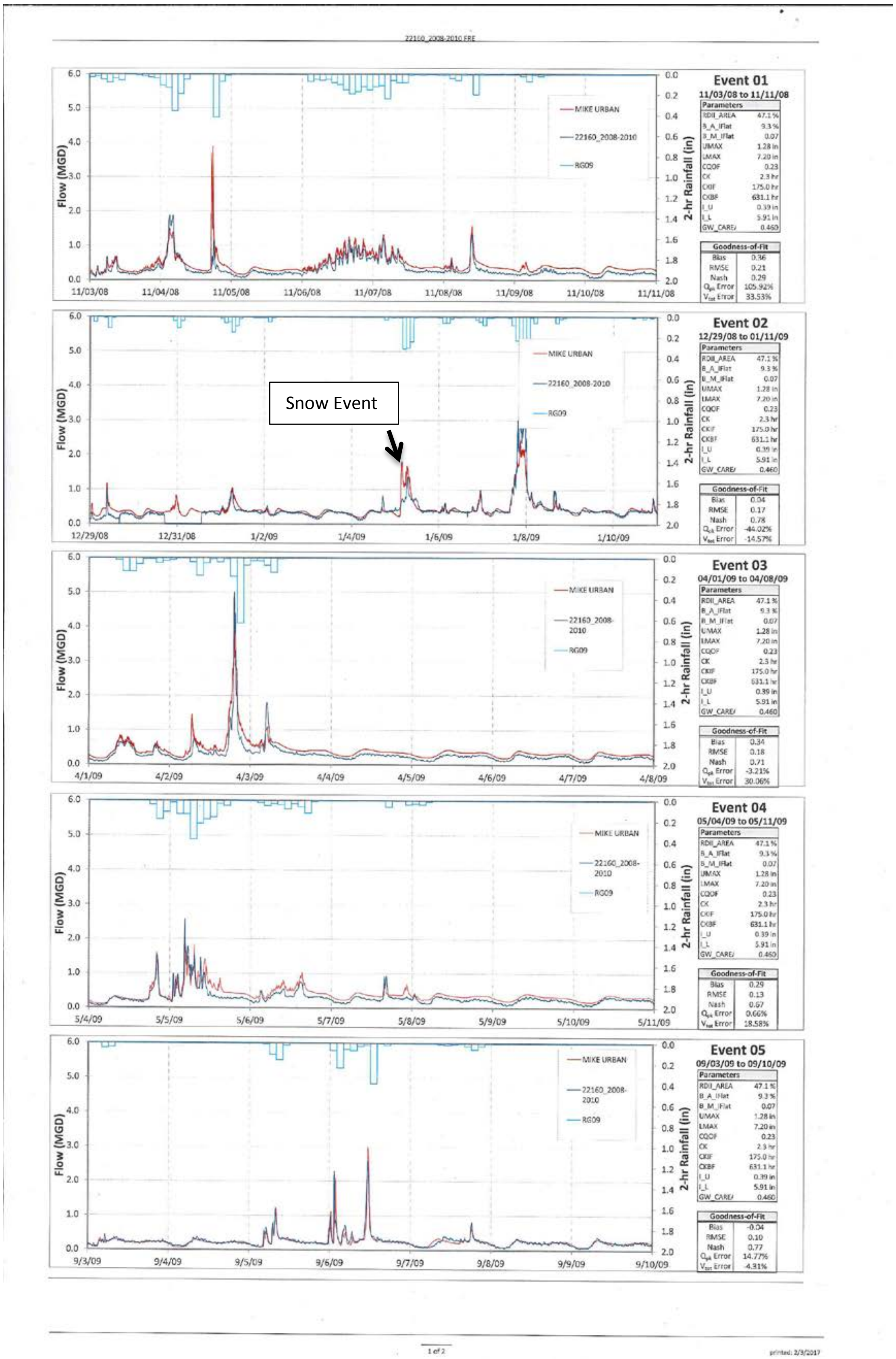


Figure 4-2: Sub-Basin 147B Hydrologic Calibration Graphs

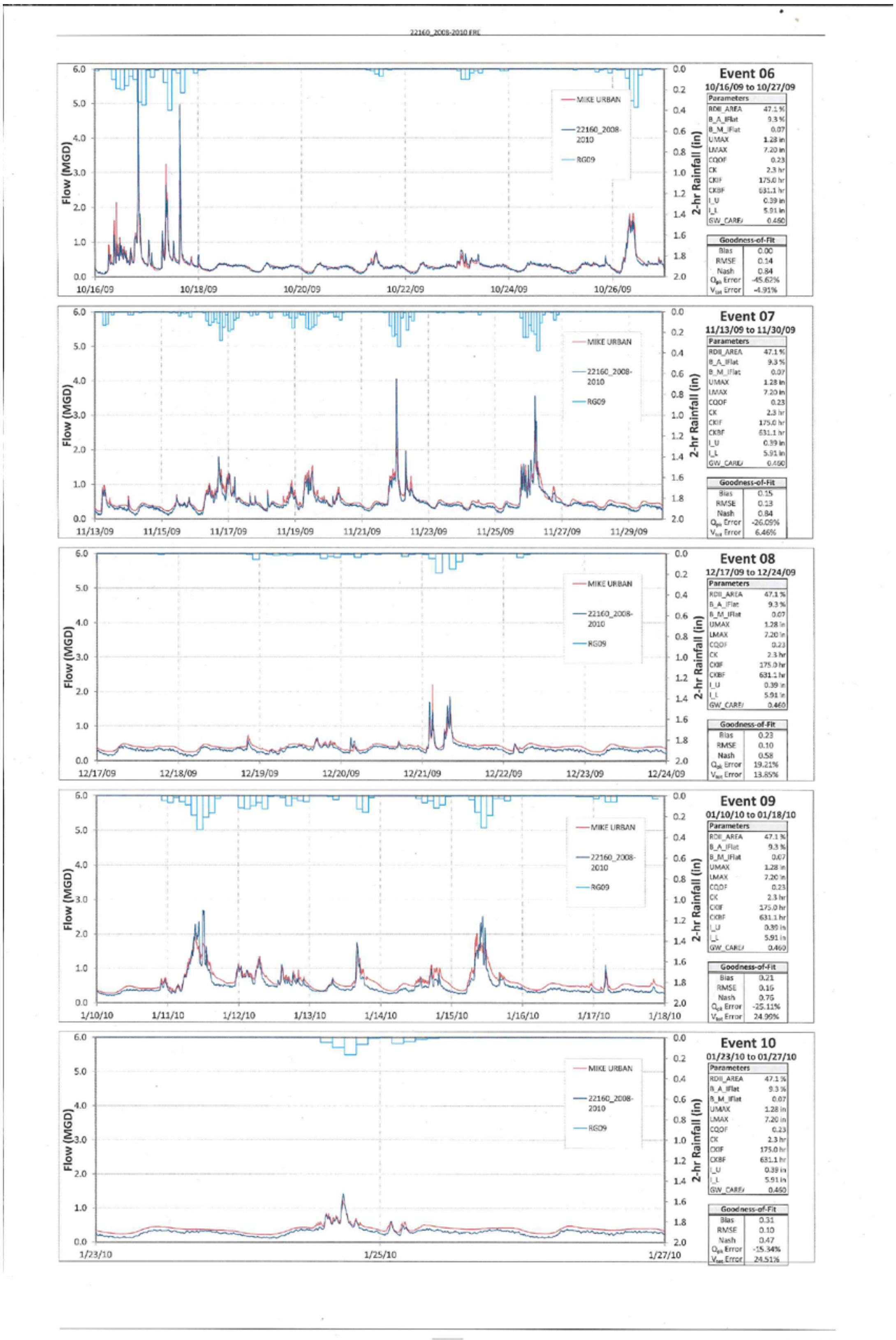


Figure 4-2 (Continued): NPDES 147B Hydrologic Calibration Graphs

4.1.1 Remarks on Hydrologic Calibration

Overall, the hydrologic calibrations for Sub-Basins 147A and 147B produce simulated hydrographs that match the measured flow data in terms of shape, peak flow rates and volumes. The Nash-Sutcliffe coefficient was generally high for the calibration event windows. The models are considered suitable for estimating flows within the system and approximating overflow volumes over the long term.

For Sub-Basin 147A, the event peak flow rate and volume was more often overestimated than underestimated with the model simulation. For Sub-Basin 147B, the simulated event peak flow rate tended to be lower than the measured while the event volume was evenly split between an over and under estimation.

Differences between simulated results and measured data are attributable, at least in part, to short duration, high intensity rainfall events in which the rainfall reported from the nearest rain gauge did not accurately represent the rainfall over the whole basin.

5 Hydraulic Calibration and Overflow Simulation Results

The primary purpose and focus of hydrologic and hydraulic model calibration was to provide for accurately simulating overflow occurrences and volumes across a range of rainfall event intensity and magnitude for these model basins. Simulated and measured overflow results presented herein include only the post-retrofit (post weir-raise) period. The reliability of the measured overflow data is higher after October 2010 when the overflow measurement location was moved to MH 022-186.

The modeled hydrographs for Sub-Basins 147A and 147B produce a reasonable match to measured overflows in terms of the number of events and simulated versus measured depths on the upstream side of the weir.

5.1 Hydraulic Calibration and Overflow Simulation Results for Sub-Basin 147A

The simulated results at the Sub-Basin 147A overflow structure were compared to observed flow meter data collected and provided to WTD by SPU. Overall the number of simulated and measured overflow occurrences match very well. The simulated overflow volumes are consistently higher than the reported volumes in terms of annual totals, as the data presented in Table 5-A show. This is likely resulting from overestimation of simulated water levels for the larger events related to inability of the model to precisely represent the complex structure hydraulics across the range of events. For this reason, post construction monitoring and adjustments to the model calibration should be considered.

The recurrence interval graph in Figure 5-1 shows that the simulated overflow volumes for most of the largest individual overflow events are consistently higher than the measured overflow volumes. Data for the ten largest measured overflow events is presented in Table 5-B. The simulated volume for the fifth largest event in the five year overflow calibration period is about 3.14 million gallons (MG) or 27 percent higher than the measured volume for that event of 2.47 MG. The largest simulated and measured overflow event during the five year calibration period occurred on November 18-21, 2012. The simulated and measured volumes for that event match very well at approximately 4.9 MG.

Comparison plots of the simulated and measured water levels on the upstream side of the weir for selected overflow events and the simulated and measured overflow hydrographs for those events are presented in Figures 5-2 through 5-9.

Table 5-A: NPDES 147A Annual Overflow Comparison - 2011-2015

Year	Metered Overflow Frequency	Modeled Overflow Frequency	Percent Error	Metered Overflow Volume (MG)	Modeled Overflow Volume (MG)	Percent Error
2011	38	34	-11%	9.7	14.2	47%
2012	48	50	4%	14.2	21.9	54%
2013	27	27	0%	4.8	7.5	58%
2014	49	46	-6%	12.2	21.1	73%
2015	31	32	3%	16.5	21.1	28%
Total:	**193**	**189**	**-2%**	**57.4**	**85.8**	**49%**

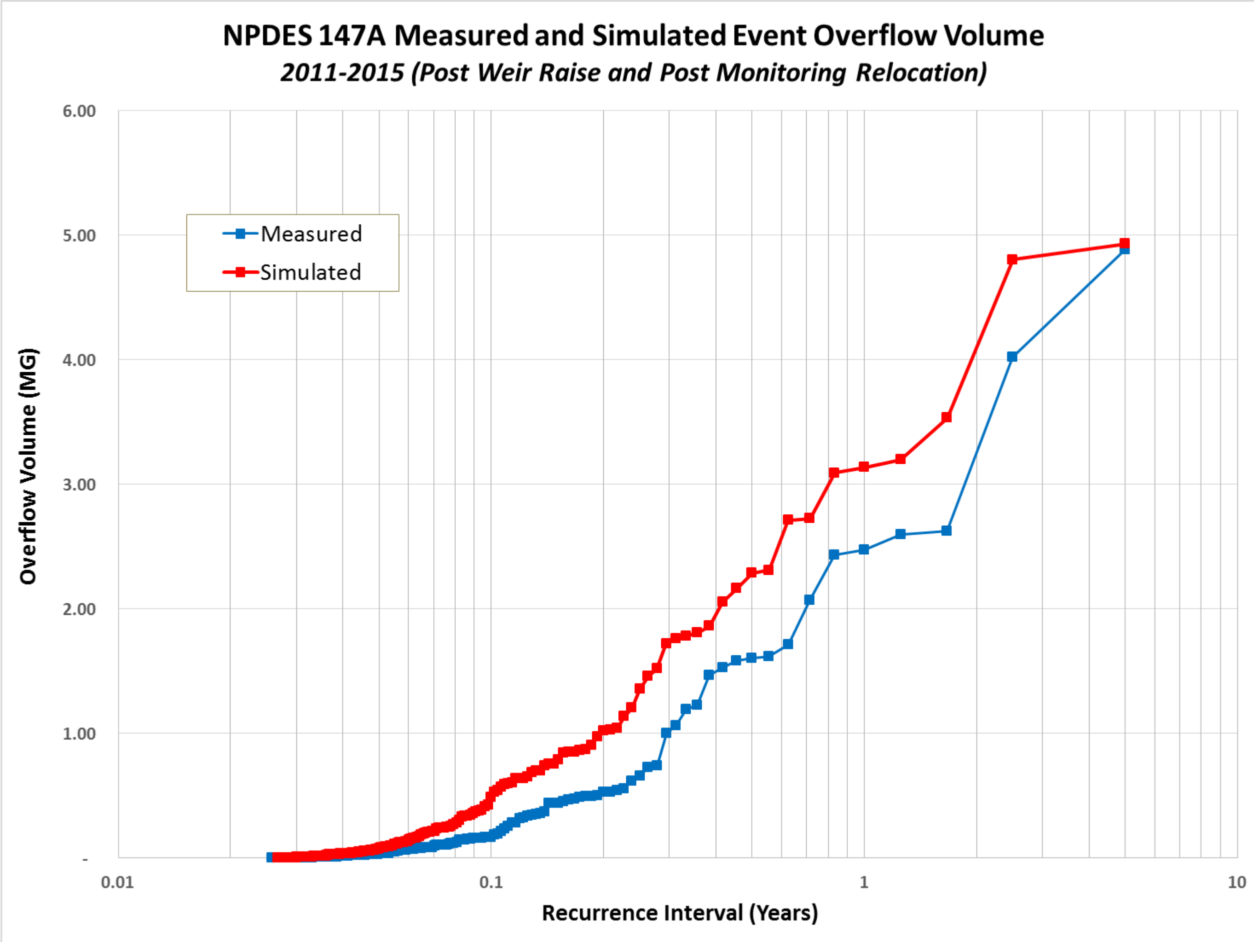


Figure 5-1: Recurrence of NPDES 147A Overflow Volumes

Table 5-B: NPDES 147A - 10 Largest Measured Overflow Volume Events - 2011-2015

Date Measured Overflow Event Begins	Recurrence Interval for Measured Overflow Event (Years)	Measured Overflow Volume (MG)	Simulated Overflow Volume (MG)	Recurrence Interval for Simulated Overflow Event (Years)
11/18/2012	5.00	4.887	4.933	5.00
12/5/2015	2.50	4.024	4.805	2.50
11/21/2011	1.67	2.623	3.094	0.83
11/30/2012	1.25	2.600	3.534	1.67
11/12/2015	**1.00**	**2.474**	**3.137**	**1.00**
3/4/2014	0.83	2.433	2.287	0.50
3/15/2015	0.71	2.074	3.200	1.25
1/12/2011	0.63	1.715	2.313	0.56
1/8/2013	0.56	1.616	2.056	0.42
3/13/2011	0.50	1.604	2.727	0.71

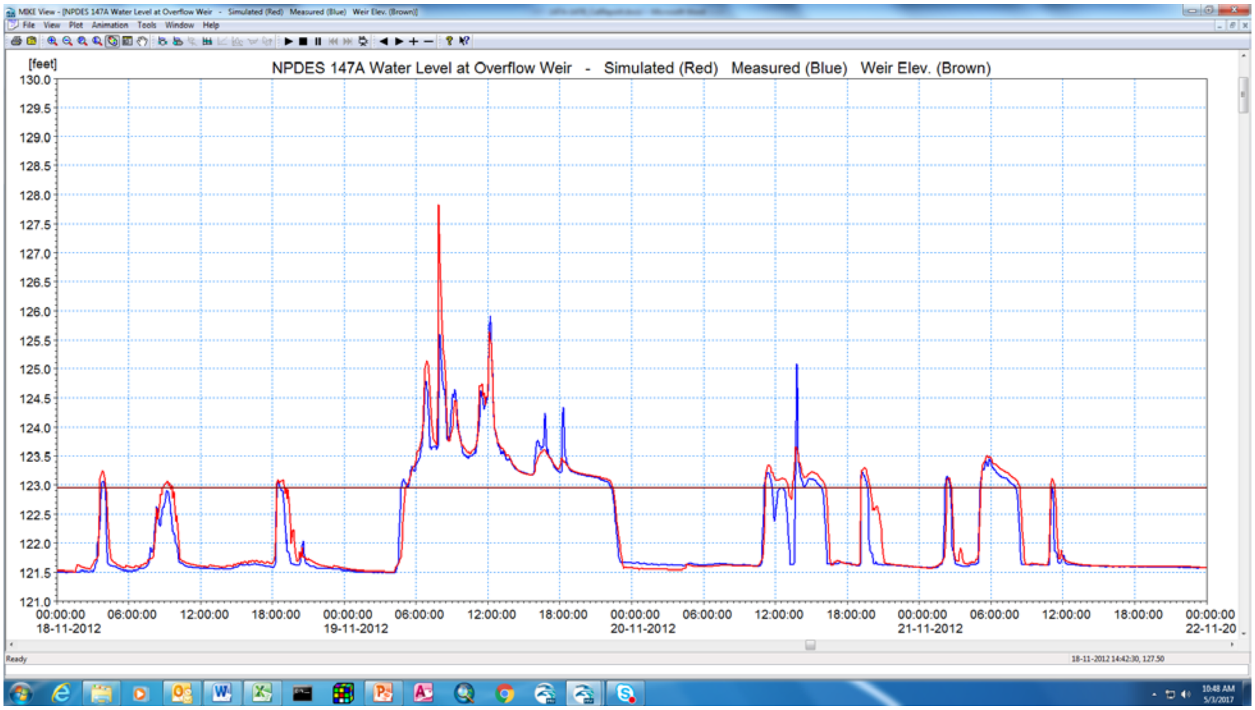


Figure 5-2: Water Level at Overflow Structure 147A - Largest Simulated and Measured Overflow Volume Event

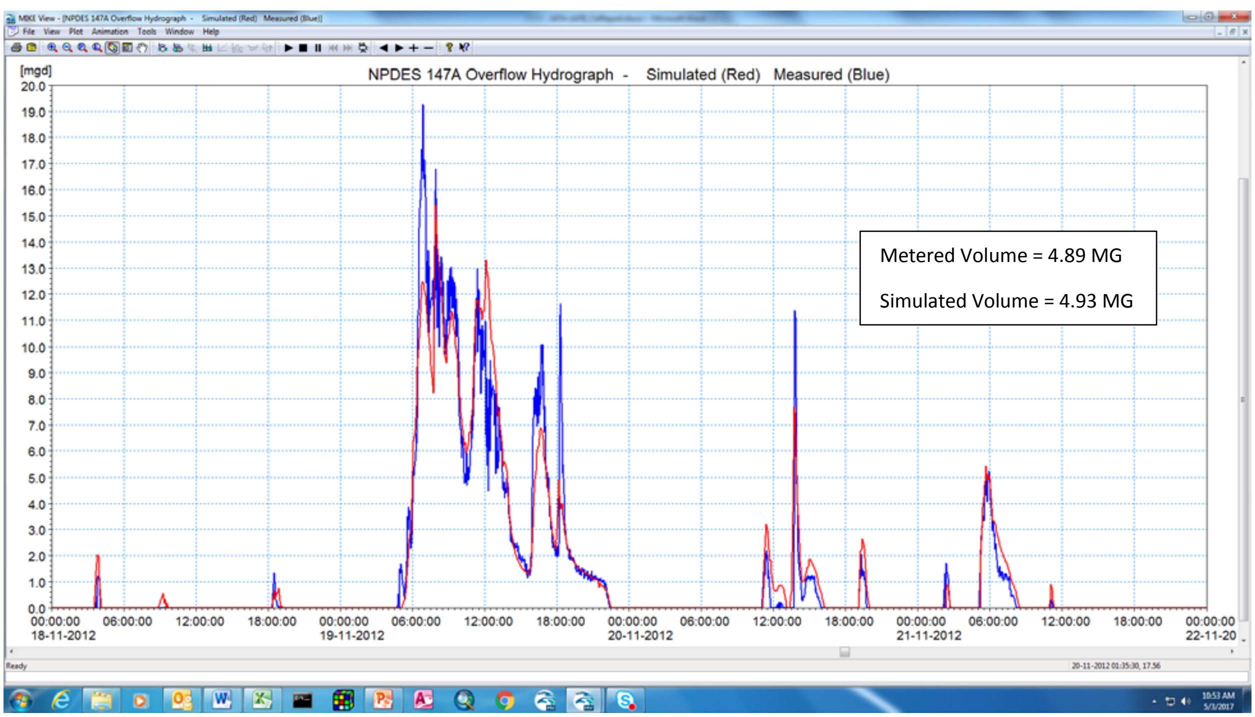


Figure 5-3: Overflow Hydrograph at Overflow Structure 147A - Largest Simulated and Measured Overflow Volume Event

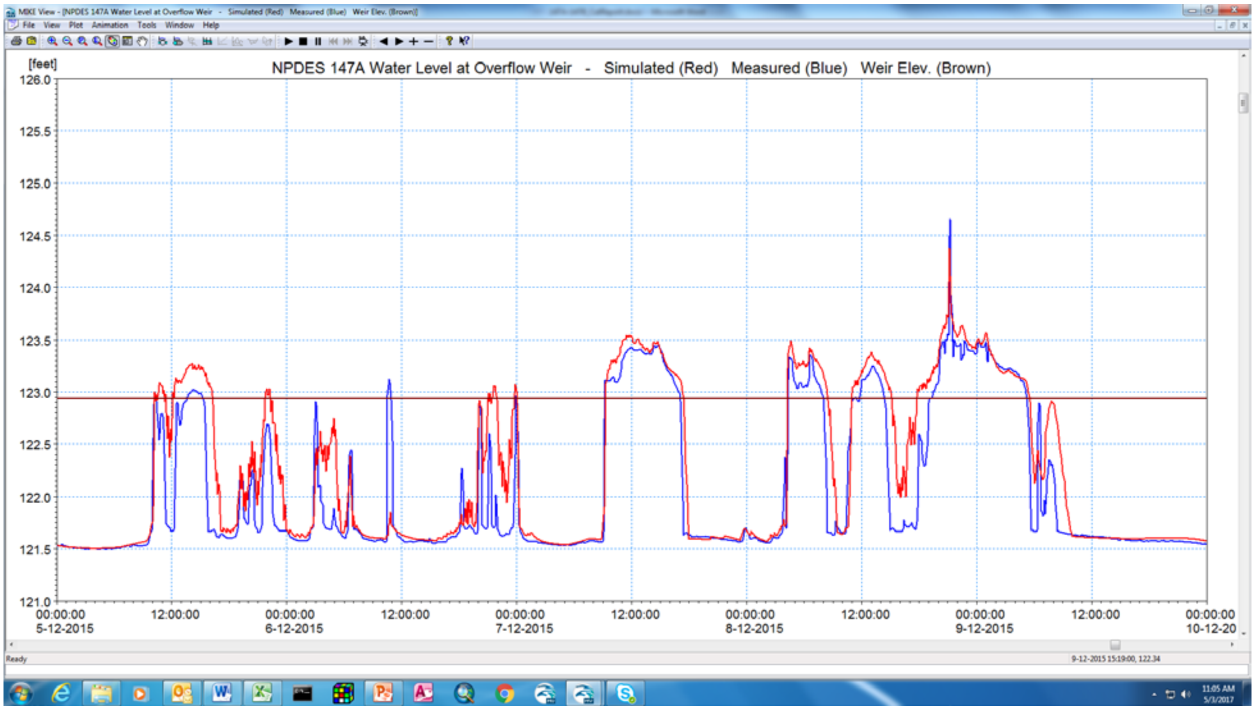


Figure 5-4: Water Level at Overflow Structure 147A - 2nd Largest Simulated and Measured Overflow Volume Event

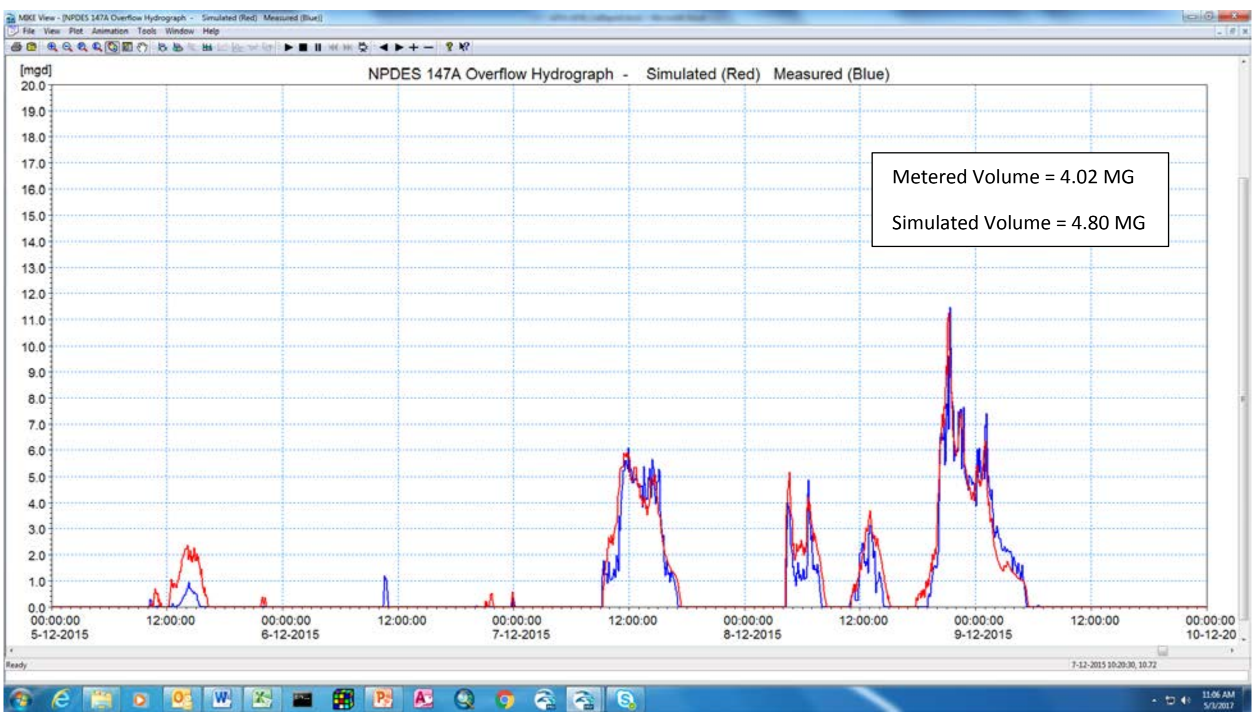


Figure 5-5: Sub-Basin 147A Overflow Hydrograph - 2nd Largest Simulated and Measured Overflow Volume Event

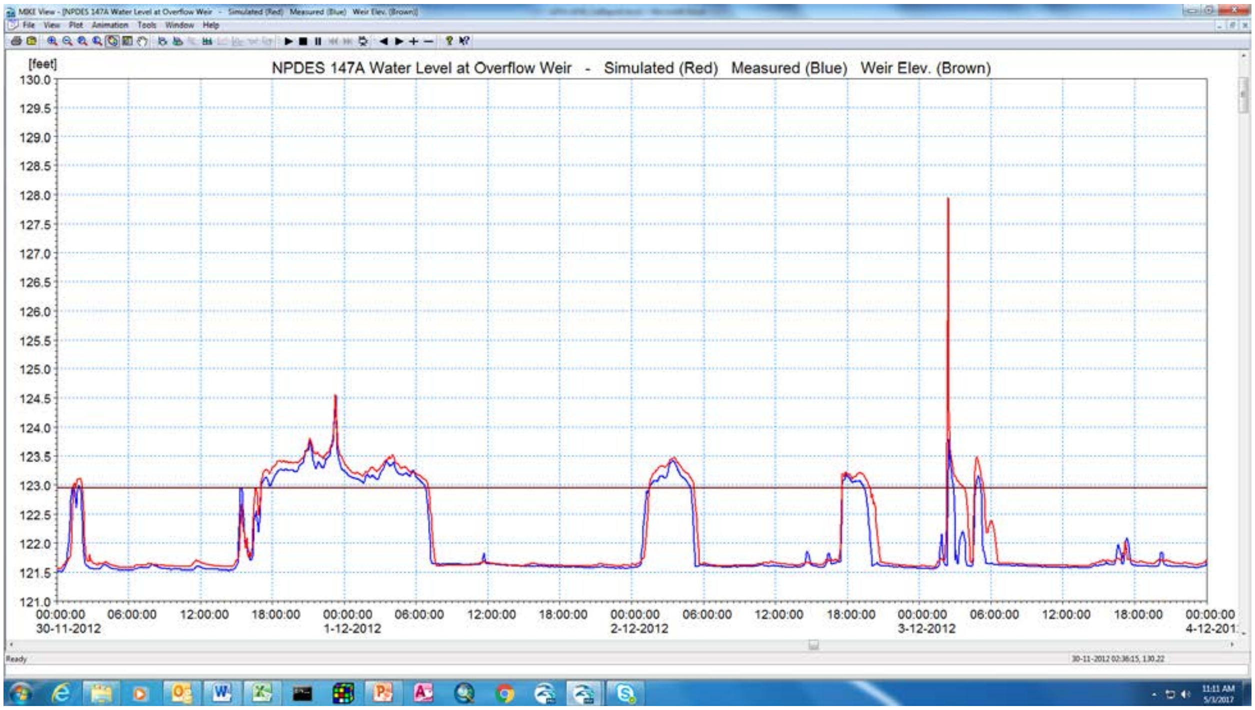


Figure 5-6: Water Level at Overflow Structure 147A - 3rd Largest Simulated Overflow Volume Event

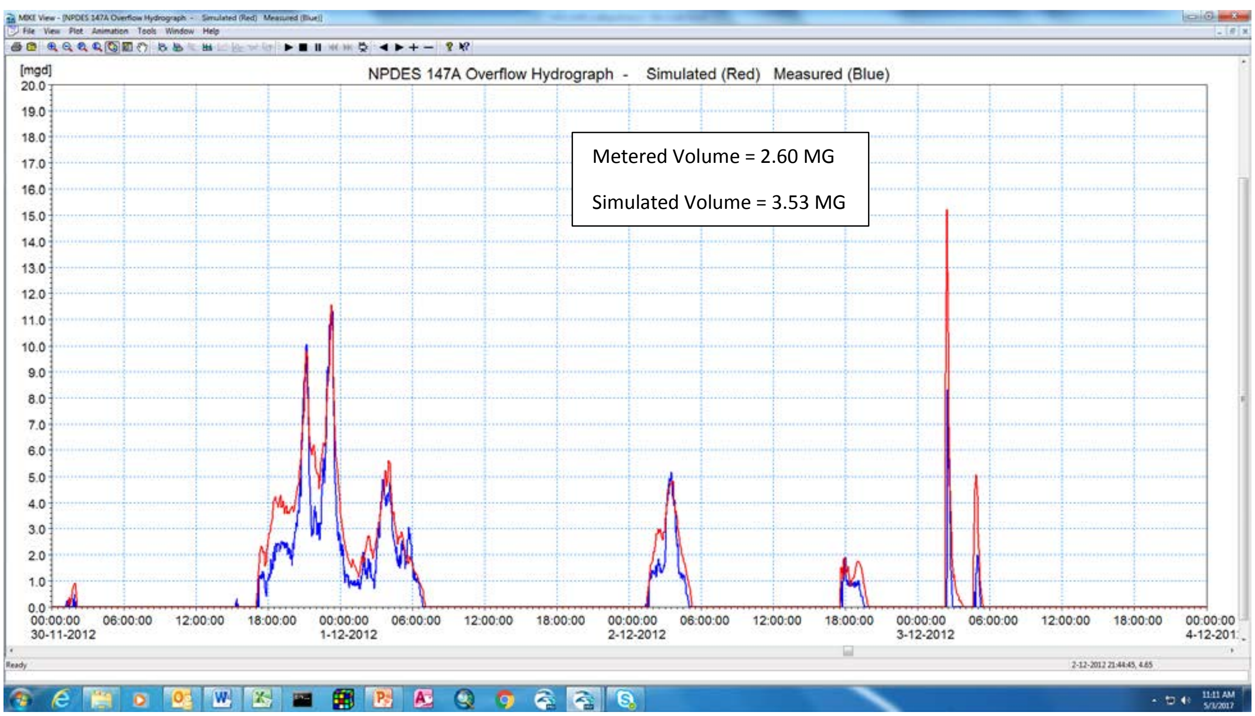


Figure 5-7: Overflow Hydrograph at Overflow Structure 147A - 3rd Largest Simulated Overflow Volume Event

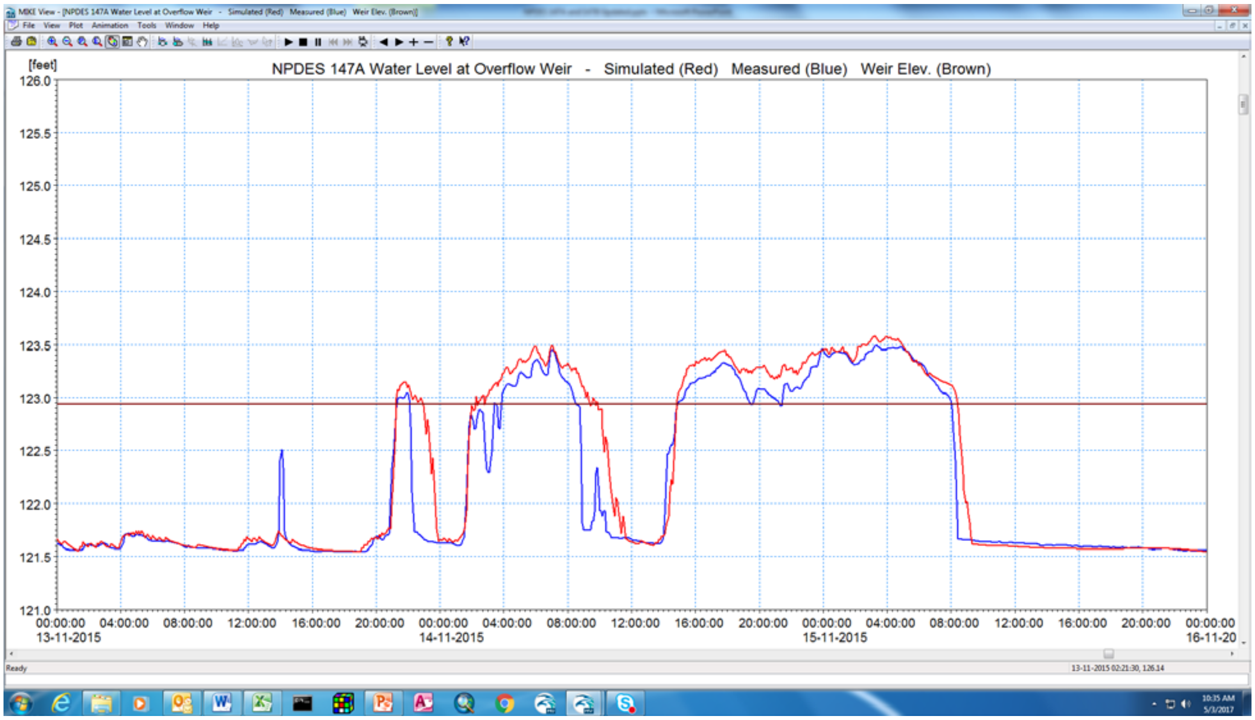


Figure 5-8: Water Level at Overflow Structure 147A - 5th Largest Simulated and Measured Overflow Volume Event

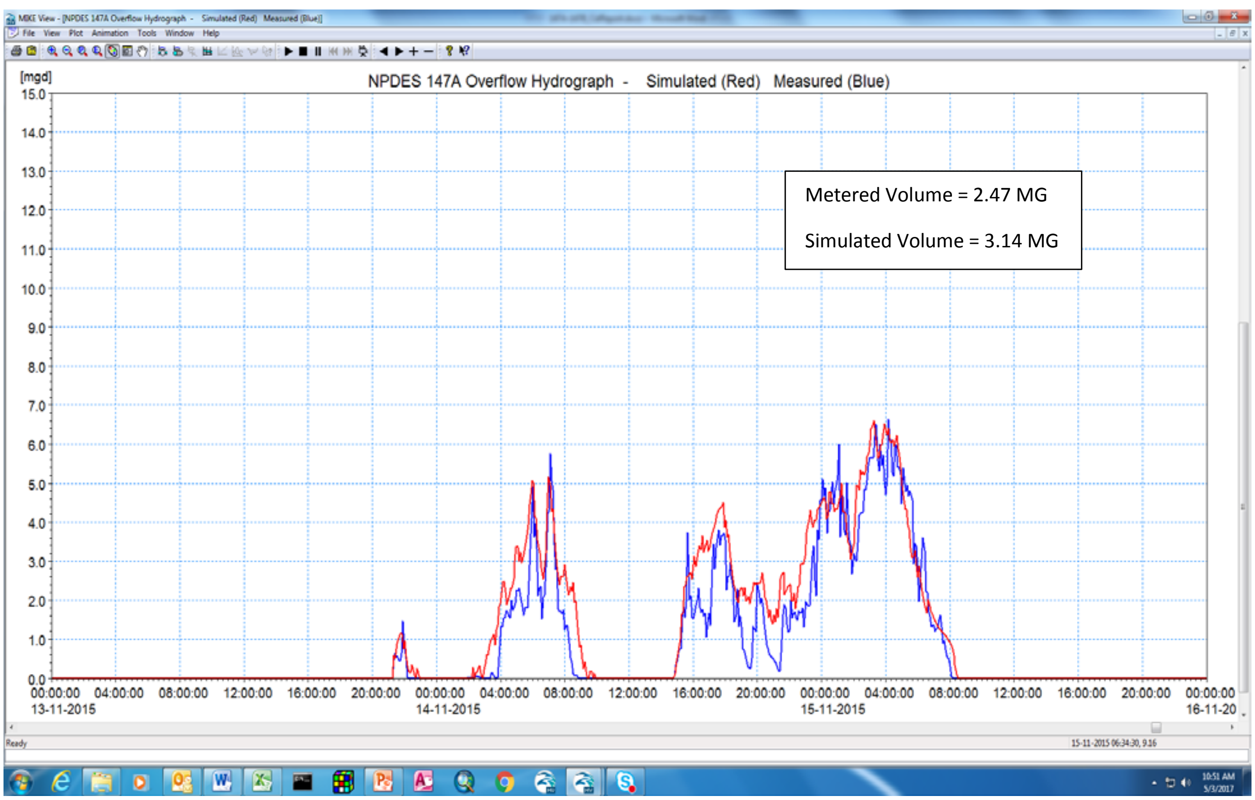


Figure 5-9: NPDES 147A Overflow Hydrograph - 5th Largest Simulated and Measured Overflow Volume Event

5.2 Hydraulic Calibration and Overflow Simulation Results for 147B

The modeled performance of the overflow structure was compared to observed flow meter data collected and provided to WTD by SPU. Overall the number of simulated and measured overflow occurrences match reasonably well as the data presented in Table 5-B show.

The recurrence interval graph in Figure 5-10 shows a reasonable match of simulated to measured overflow volumes for most of the largest individual overflow events. Data for the ten largest measured overflow events is presented in Table 5-D. The volume for the fifth largest simulated overflow event in the five year overflow calibration period is about 0.043 million gallons (MG) compared with the measured volume for that event of 0.037 MG. The largest simulated and measured overflow event during the 5-year calibration period occurred on November 19, 2012. The simulated and measured volumes for that event match very well at 0.35 MG.

Comparison plots of the simulated and measured water levels on the upstream side of the weir for selected overflow events and the simulated and measured overflow hydrographs for those events are presented in Figures 5-11 through 5-22.

Table 5-C: NPDES 147B Annual Overflow Comparison – 2011-2015

Year	Metered Overflow Frequency	Modeled Overflow Frequency	Percent Error	Metered Overflow Volume (MG)	Modeled Overflow Volume (MG)	Percent Error
2011	5	4	-20%	0.09	0.04	-56%
2012	4	3	-25%	0.39	0.37	-5%
2013	2	4	100%	0.04	0.04	0%
2014	4	5	25%	0.07	0.16	129%
2015	5	5	0%	0.17	0.21	24%
Total:	**20**	**21**	**5%**	**0.76**	**0.82**	**8%**

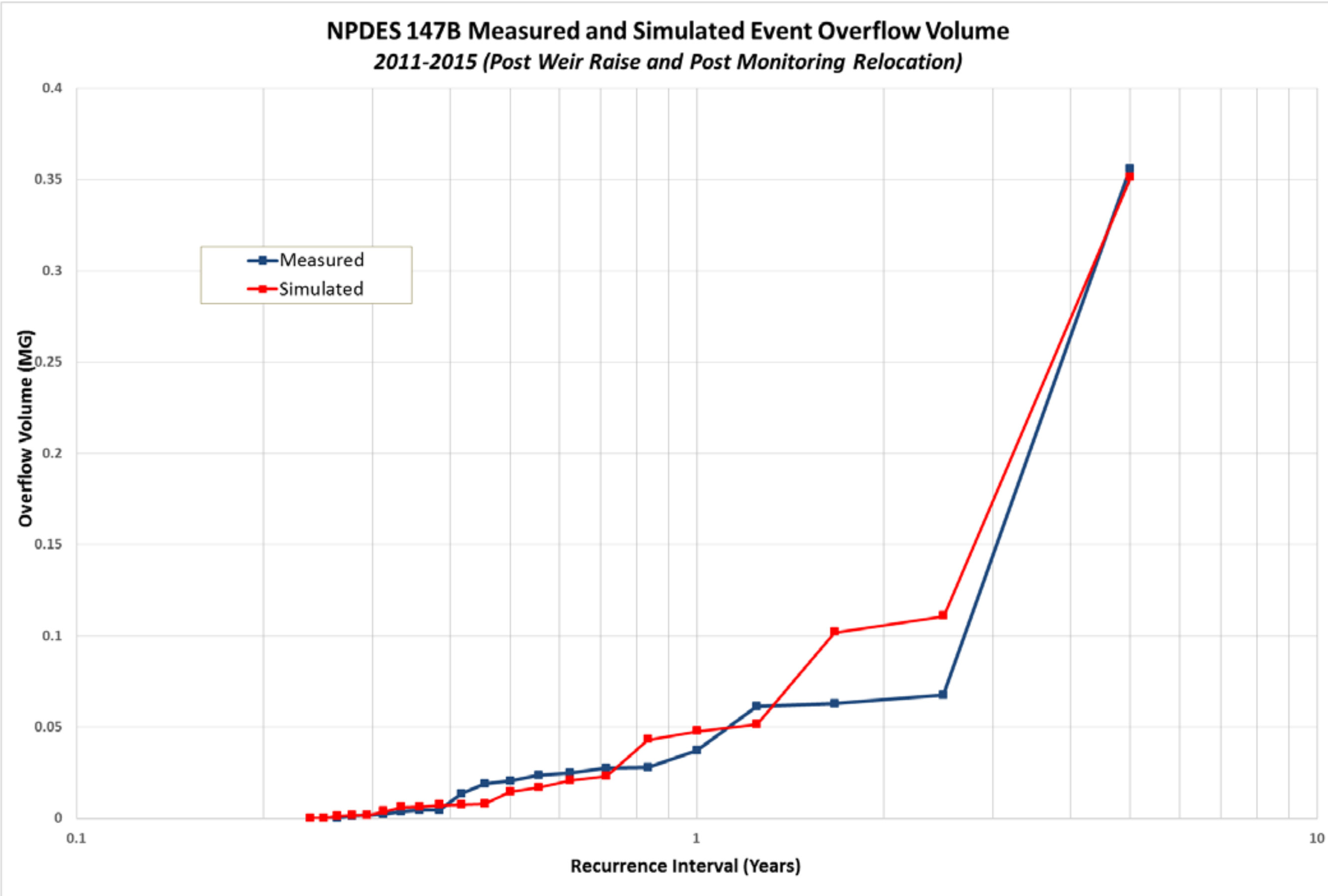


Figure 5-10: Recurrence of NPDES 147B Overflow Volumes

Table 5-D: NPDES 147B - 10 Largest Measured Overflow Volume Events - 2011-2015

Date Measured Overflow Event Begins	Recurrence Interval for Measured Overflow Event (Years)	Measured Overflow Volume (MG)	Simulated Overflow Volume (MG)	Recurrence Interval for Simulated Overflow Event (Years)
11/19/2012	5.00	0.356	0.351	5.00
12/10/2015	2.50	0.068	0.048	1.00
3/5/2014	1.67	0.063	0.102	1.67
3/15/2015	1.25	0.061	0.111	2.50
12/8/2015	1.00	0.037	0.043	0.83
3/14/2011	0.83	0.028	0.017	0.56
9/6/2013	0.71	0.028	0.023	0.71
11/30/2012	0.63	0.025	0.021	0.63
1/12/2011	0.56	0.024	0.014	0.50
3/9/2011	0.50	0.021	< 0.001	0.25

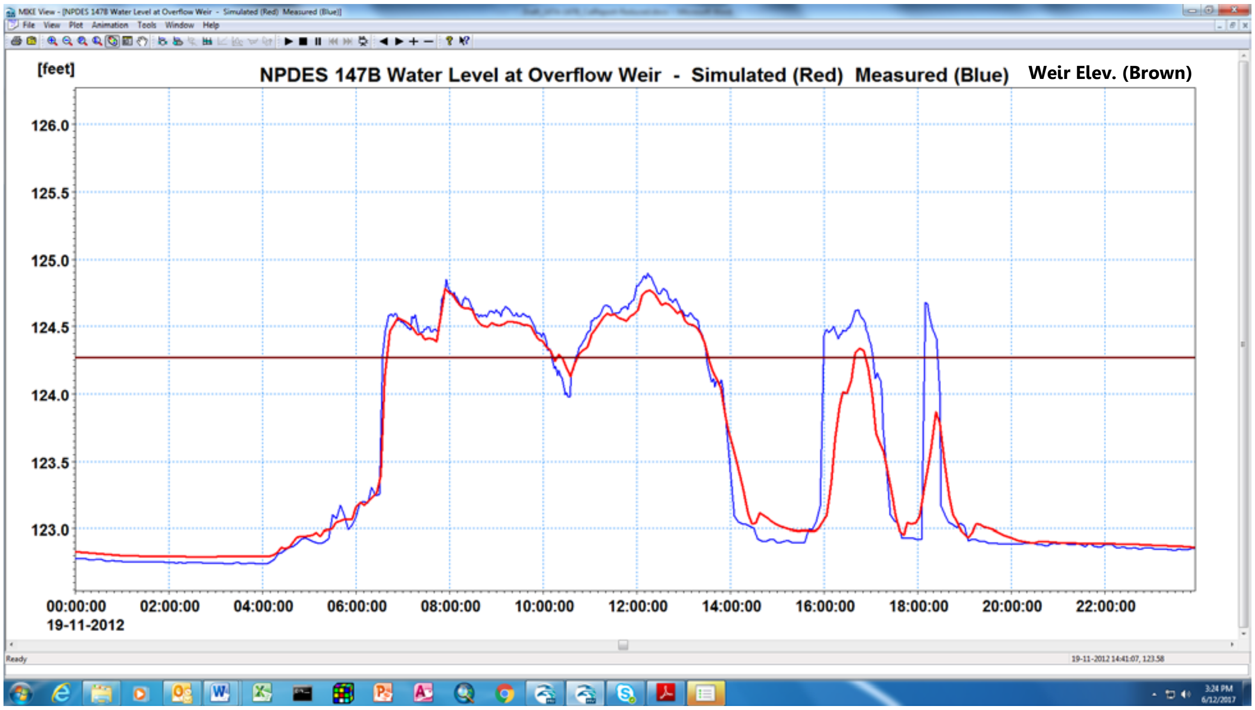


Figure 5-11: Water Level at NPDES 147B Overflow Weir - Largest Overflow Volume Event

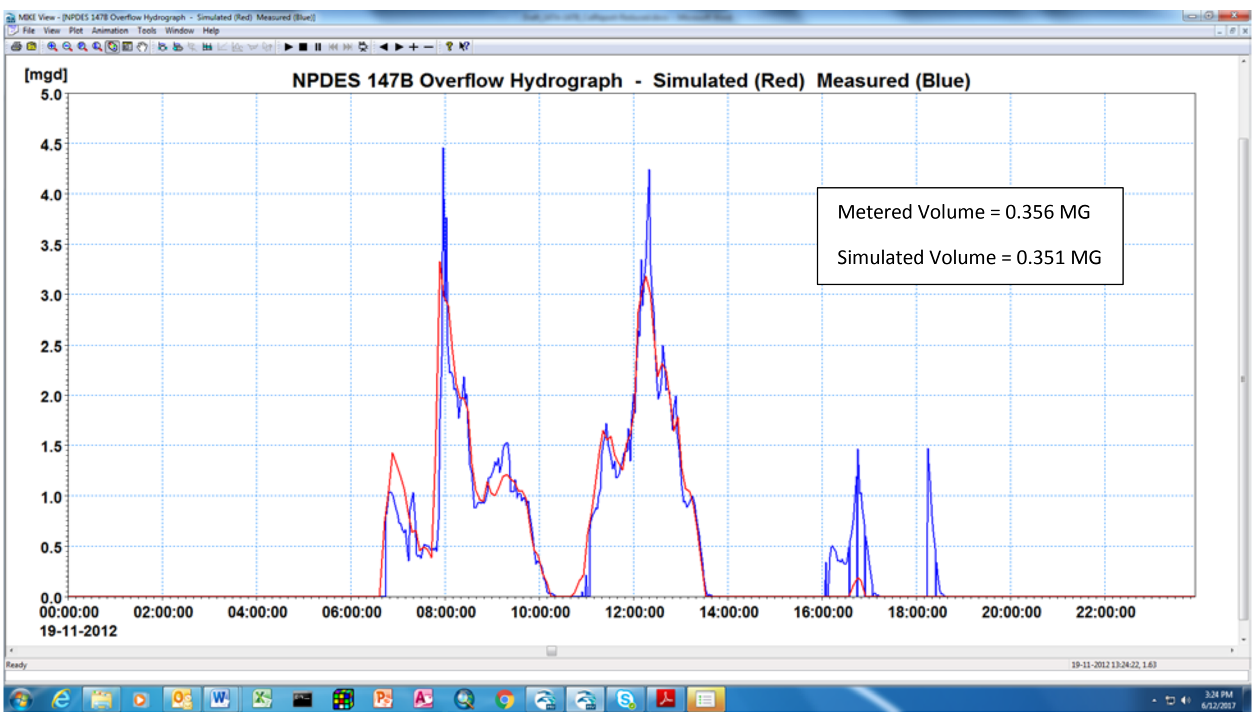


Figure 5-12: Overflow Hydrograph at NPDES 147B Overflow Weir - Largest Overflow Volume Event

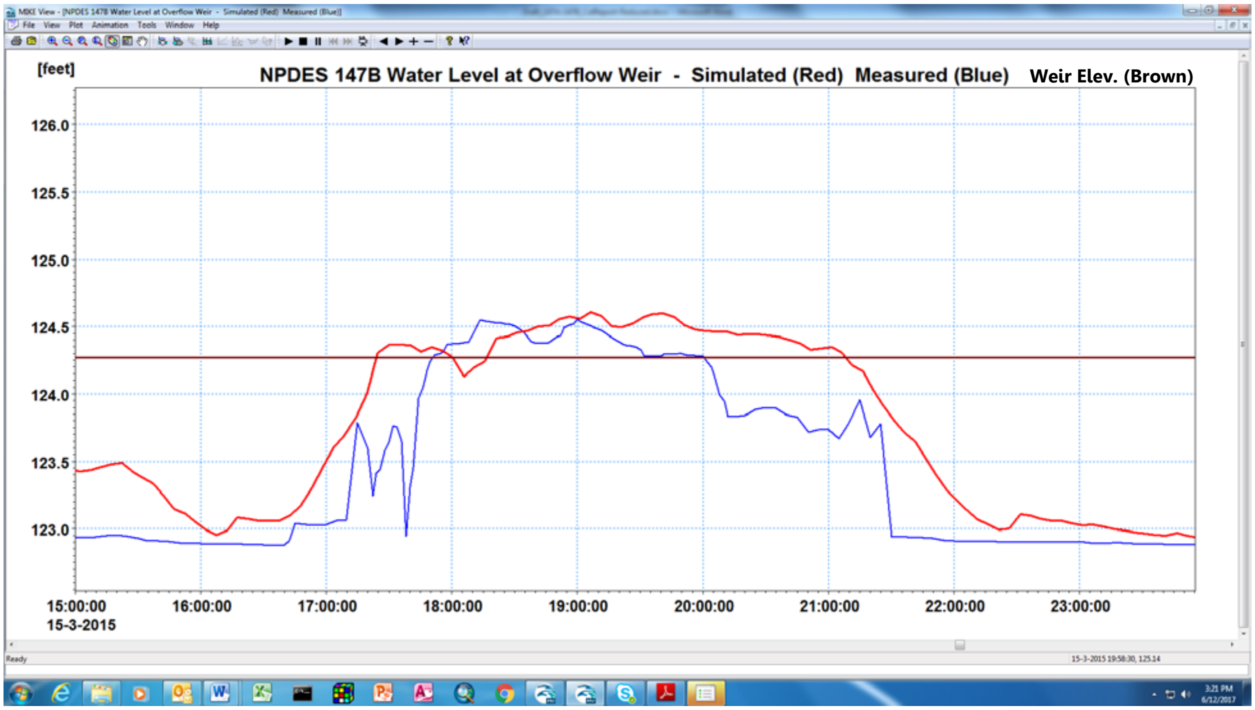


Figure 5-13: Water Level at NPDES 147B Overflow Weir – Second Largest Overflow Volume Event 2011-2015

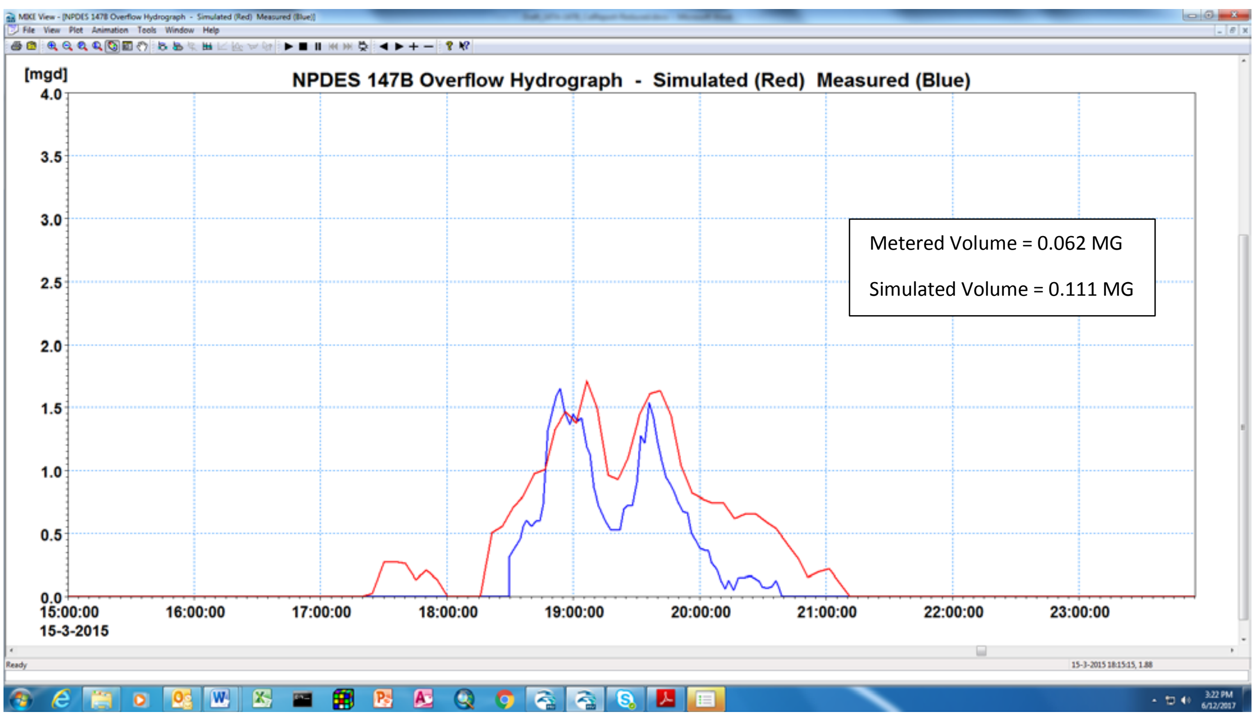


Figure 5-14: Overflow Hydrograph at NPDES 147B Overflow Weir - Second Largest Overflow Volume Event

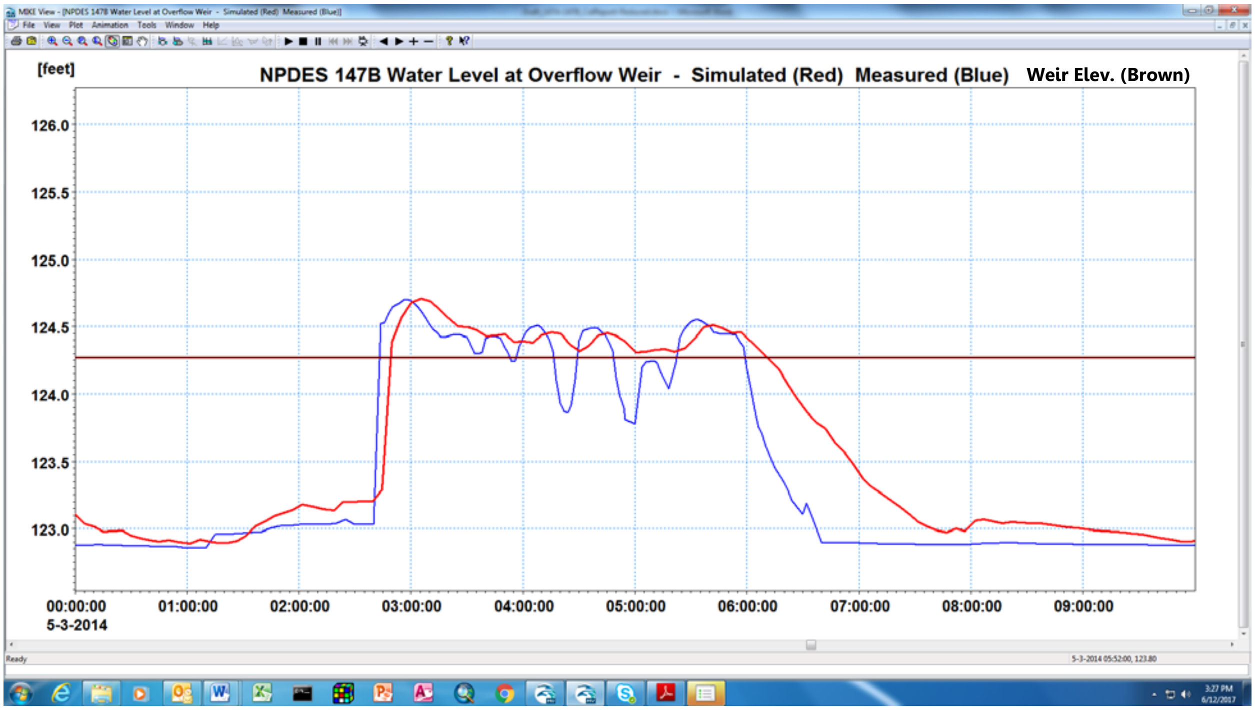


Figure 5-15: Water Level at NPDES 147B Overflow Weir – Third Largest Overflow Volume Event 2011-2015

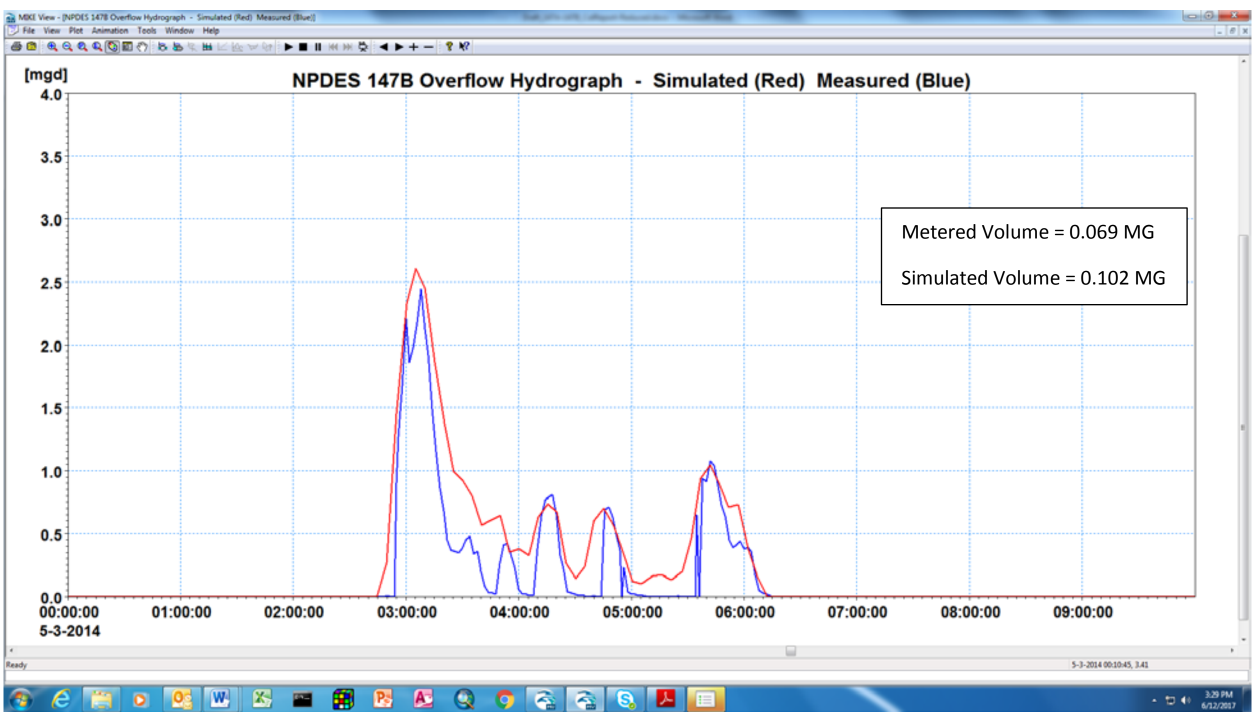


Figure 5-16: Overflow Hydrograph at NPDES 147B Overflow Weir - Third Largest Overflow Volume Event

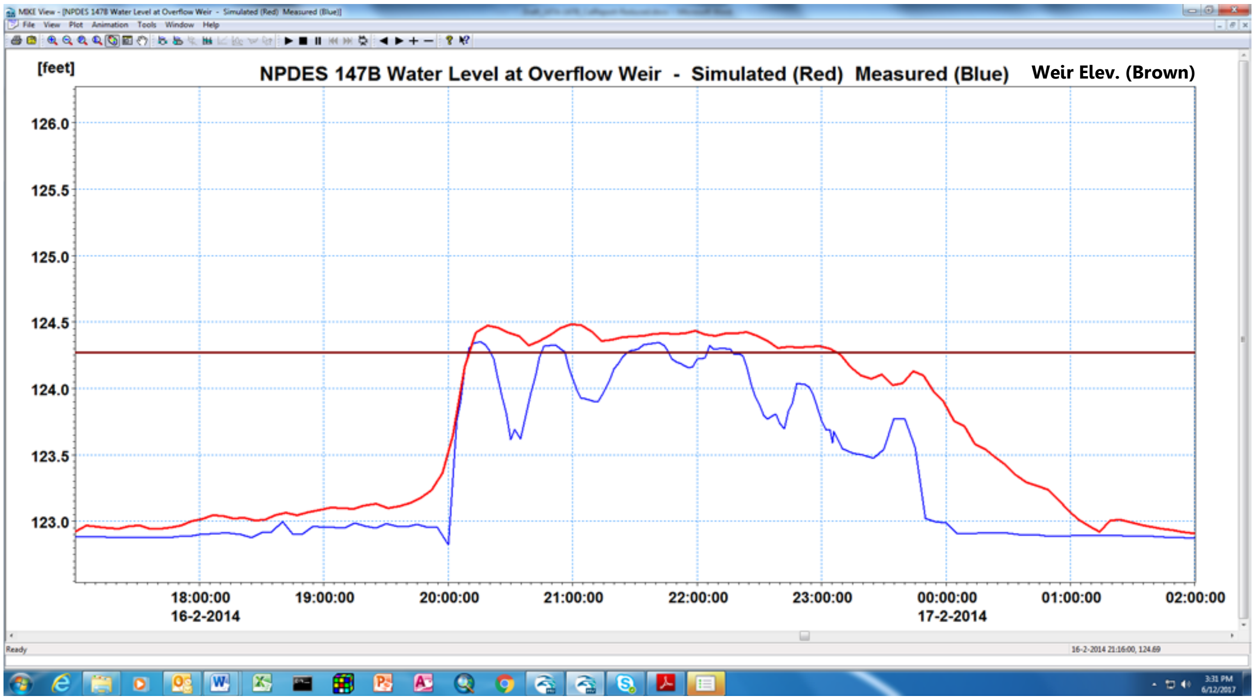


Figure 5-17: Water Level at NPDES 147B Overflow Weir – Fourth Largest Overflow Volume Event 2011-2015

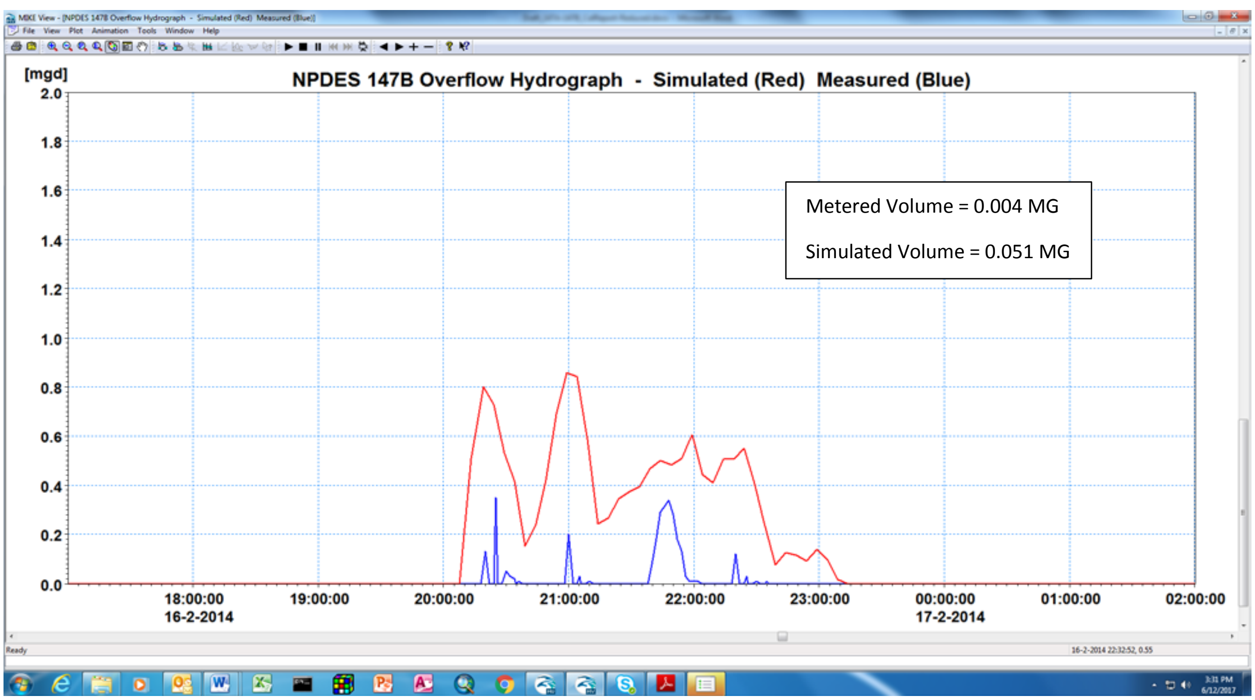


Figure 5-18: Overflow Hydrograph at NPDES 147B Overflow Weir - Fourth Largest Overflow Volume Event

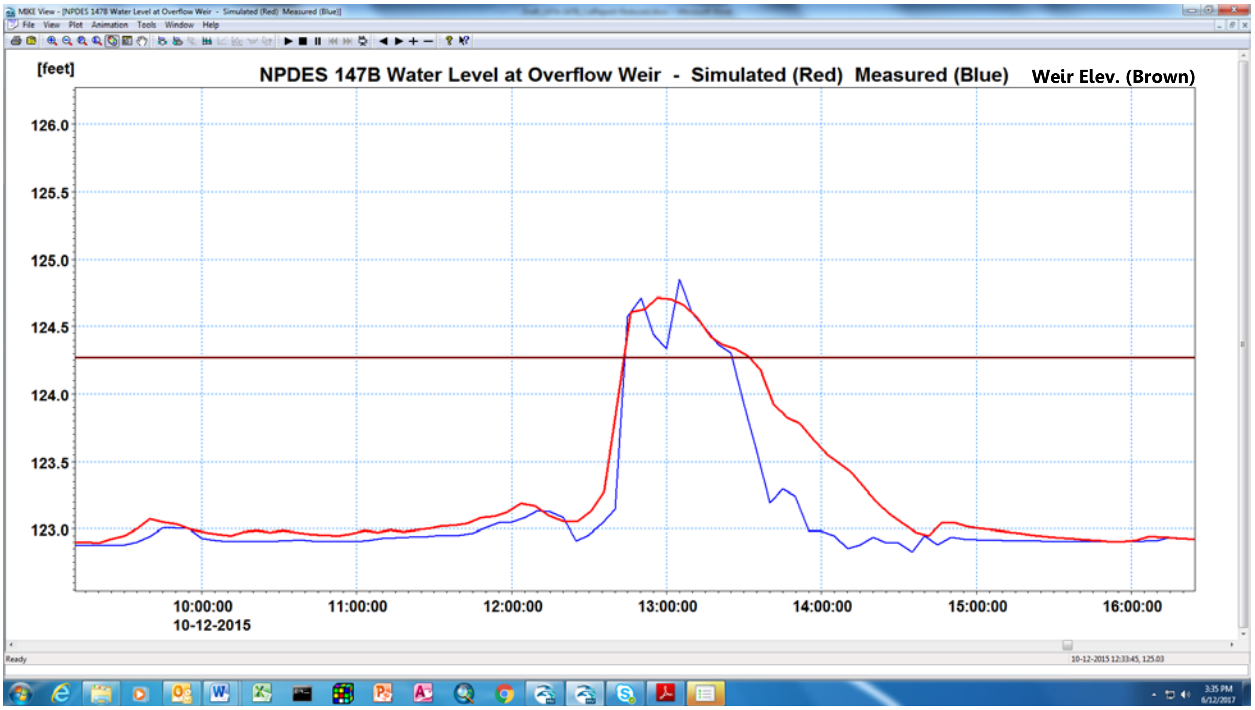


Figure 5-19: Water Level at NPDES 147B Overflow Weir – Fifth Largest Overflow Volume Event 2011-2015

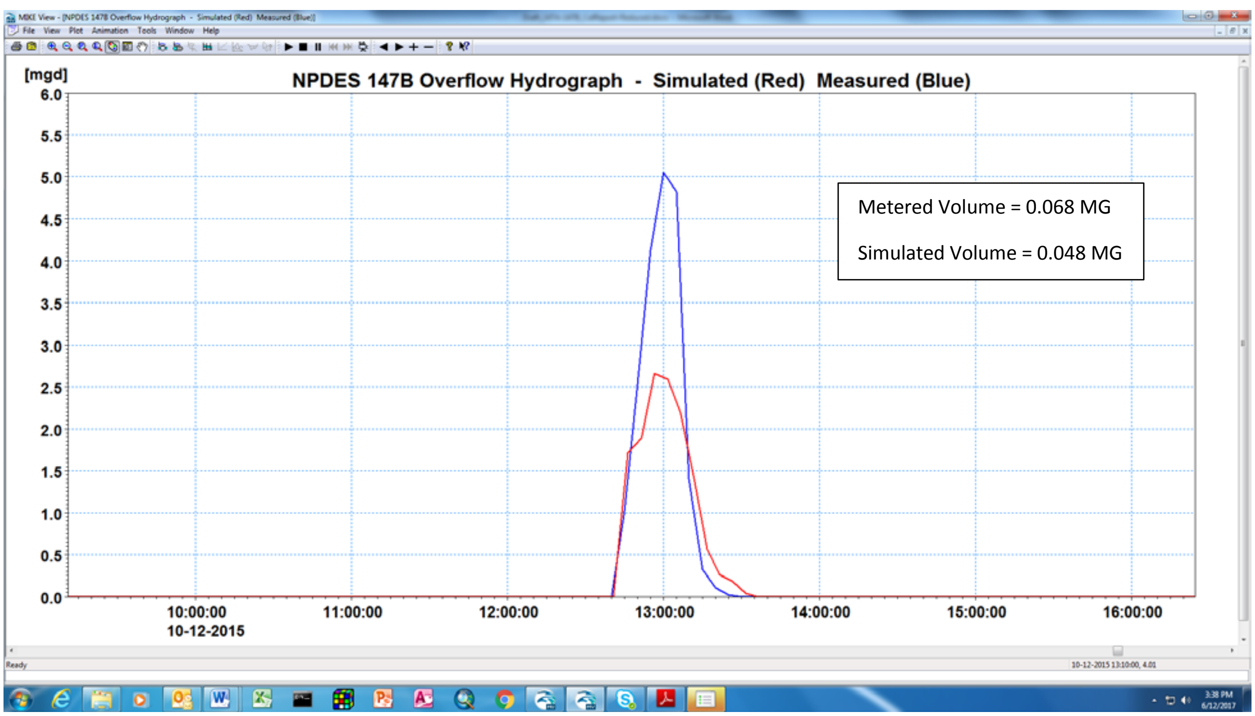


Figure 5-20: Overflow Hydrograph at NPDES 147B Overflow Weir - Fifth Largest Overflow Volume Event

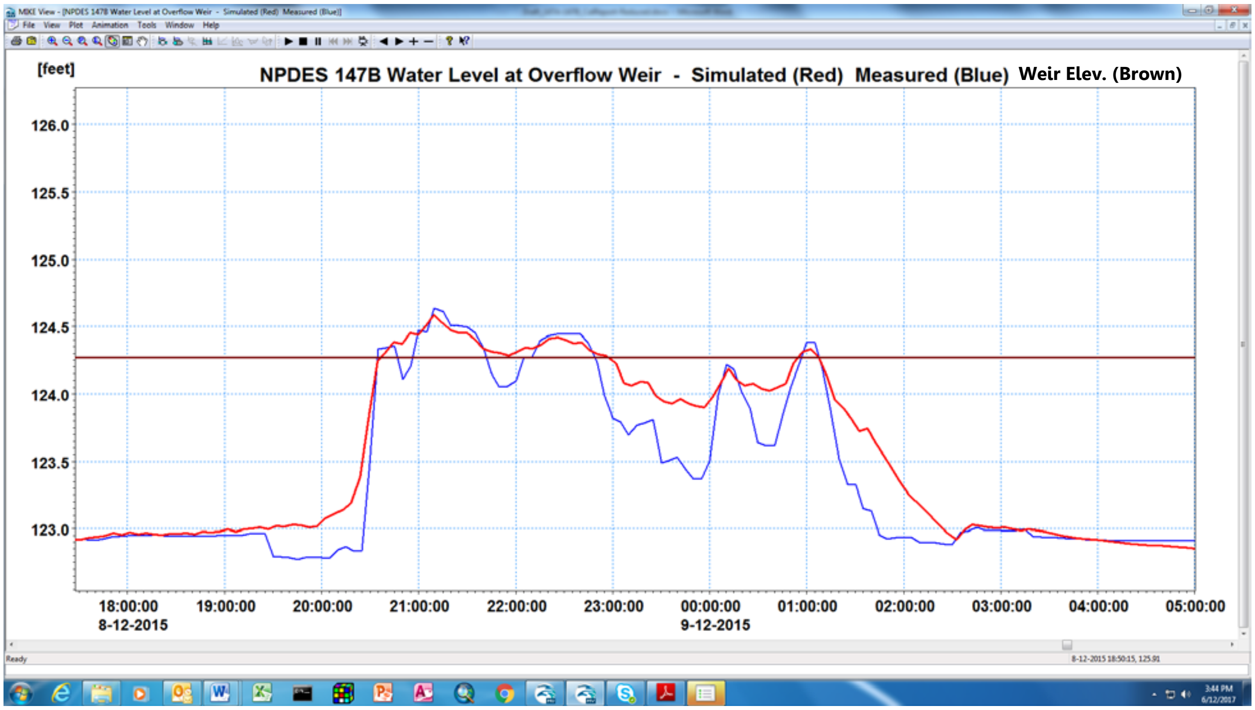


Figure 5-21: Water Level at NPDES 147B Overflow Weir – Sixth Largest Overflow Volume Event 2011-2015

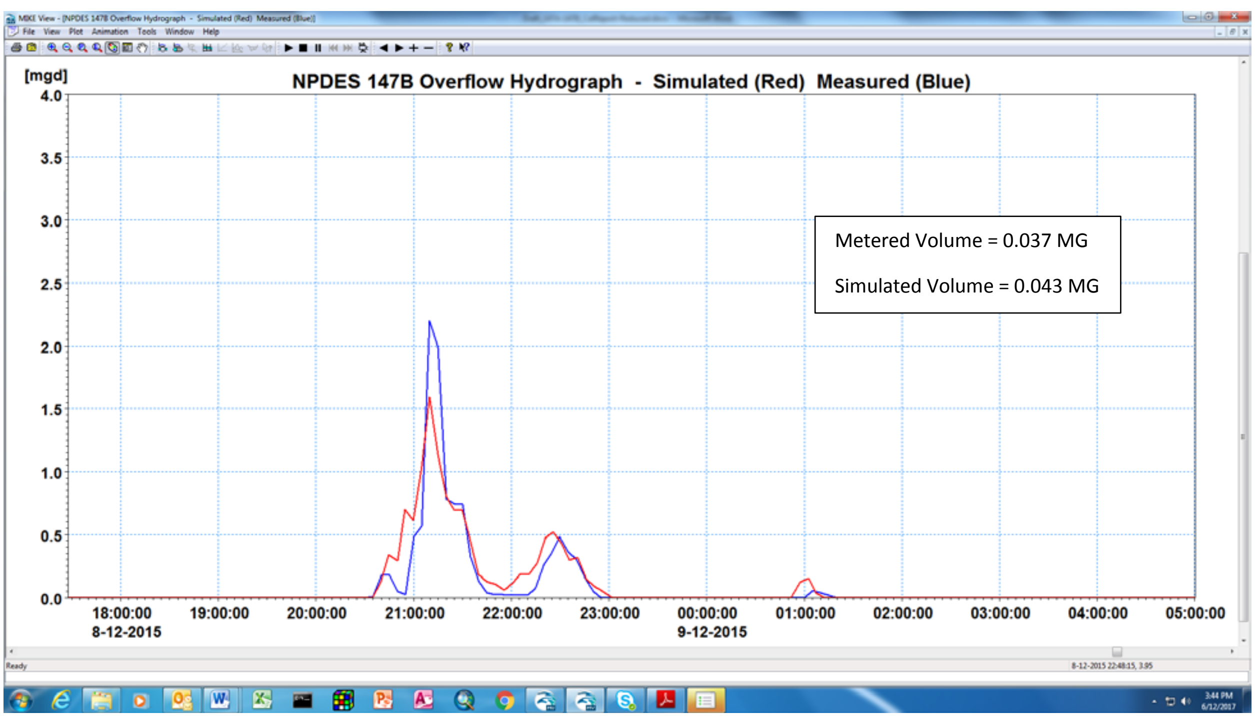


Figure 5-22: Overflow Hydrograph at NPDES 147B Overflow Weir - Sixth Largest Overflow Volume Event

Appendix B. Fremont Basin 174 Model Development and Calibration Report

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Fremont Basin 174 Model Development and Calibration Report

John Conway

January 16, 2018

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Table of Figures and Tables

List of Abbreviations

CSO	Combined Sewer Overflow
CSS	Combined Sewer System
DHI	Danish Hydraulics Institute
DWF	Dry Weather Flow
FRC	Fast Response Component
GIS	Geographic Information System
KC	King County
MGD	Million gallons per day
MH	Manhole
NPDES	National Pollutant Discharge Elimination System
RDII	Rainfall-Dependent Infiltration and Inflow
RMSE	Root-Mean-Square Area
RTC	Real Time Control
SCADA	Supervisory Control And Data Acquisition
SCWQP	Ship Canal Water Quality Project
SPU	Seattle Public Utilities
SRC	Slow Response Component
WTD	Wastewater Treatment Division

1 Introduction

This report documents the continued development and calibration of hydrologic and hydraulic models for the City of Seattle in the Fremont area (Basin 174) and its downstream connection to King County's North Interceptor. Fremont Basin 174 overflows discharge into Salmon Bay (Lake Washington Ship Canal) near 2nd Ave NW and NW Canal St. Outfall 174 averaged 14 combined sewer overflow (CSO) events per year from 2002 - 2016. .SPU previously developed EPA SWMM5 hydrologic and hydraulic models as part of their 2015 LTCP. Information gained and lessons learned from that modeling effort were used in the current effort to develop and calibrate a model of the basin and overflow structure using the municipal wastewater modeling software MIKE URBAN, developed and distributed by the Danish Hydraulics Institute (DHI). MOUSE is the hydrologic and hydraulic engine within the Mike Urban software. The results of this modeling effort contributed to the development of an Integrated Model with the King County North Interceptor and will contribute to planning, design, construction and operation of the joint Ship Canal Water Quality Project.

1.1 Model Development Approach

The area upstream of Basin 174 has modeled as 2 sub-basins, NPDES174 and NPDES148. These sub-basins are used as hydrologic modeling units and they were calibrated using quality checked data from a unique portable flow meter. These sub-basins were imported into the municipal wastewater modeling software MIKE URBAN, developed and distributed by the Danish Hydraulics Institute (DHI), where they were assigned hydrologic parameters and connected to the network of pipes, manholes, and control structures in the area. The downstream flow meter data was then used to calibrate the hydrologic parameters of the sub-basins together, a process automated using the calibration program PEST. Section 4 will describe the hydrologic calibration process in greater detail.

1.2 Service Area and Operation

The Basin 174 CSO Area encompasses 386 acres of Seattle. It is bounded by the Lake Washington Ship Canal to the south and southwest; 7th Ave NW to the west; wrapping around NW 45th St, Phinney Ave N, Woodland Park zoo, and Fremont Ave N; and approximately Linden Ave N to the east. Basin 174 and Basin 148 are both considered partially separated, meaning that the stormwater is, in some areas, conveyed via a different pipe than is the sanitary sewage and does not enter the combined sewer.

The area's hydraulic model was limited to reduce run time and eliminate pieces nonessential to the goals of the modeling effort. The Basin 174 model represents solely the conveyance necessary to a) calibrate hydrologically using a flow meter; b) represent the hydraulic structures that impact overflow events, including the flap gate between the overflow weir and the North Interceptor, c) simulate the impacts of the North Interceptor water level on the overflow rates and volumes, and d) accurately allocate flow to the North interceptor for long term time series. The downstream boundary of the model is the water level in the North Interceptor. Section 3.1 discusses both sub-basins in greater detail, and Section 5.1 discusses the Basin 174 Overflow weir and flap gate in greater detail.

2 Data Sources

Data was collected from a variety of sources to develop the hydrologic and hydraulic models. Hydrology for the basin was characterized from photography, contour data, evaporation, rainfall, and existing delineations of hydrologic basins (from King County's *Runoff-transport* model). Hydraulics for the collection system were defined based on as-built drawings, GIS sewer coverages, SCADA, and flow meters. Additionally, portions of the SPU modeling report were used as secondary data sources or to verify inferences and approximations.

2.1 Contour Data

GIS overlays of contour data were used to qualitatively estimate the slope of pipe systems within Basin 174. Due to the generally uniform slope of the basin, this was generally only used as an order of magnitude verification of the survey data for the pipes and manholes.

2.2 GIS

Point-based shape files of sewer manholes, and line-based files of sewer pipes, were the primary source of the network data used to parametrize the hydraulic model network. Manhole data includes name, diameter of the pipe intersected, and elevations of the rim and of each inlet and outlet pipe. Pipe data include length, diameter, upstream and downstream invert elevations, and material. Polygon-based shapefiles of KC-WTD CSO basins provided the foundation for the basins used in the hydrologic model, and ArcMap was used to create sub-basins based on flow directions as well as to calculate the area of each sub-basin.

2.3 Evaporation

A long-term evaporation record was downloaded from the AgWeatherNet Washington State University Puyallup site. This record was averaged by month into an average evaporation year, which was repeated for the duration of the calibration and long-term period of simulation. Both this data source and methodology are commonly used for continuous hydrologic modeling in the Puget Sound area.

2.4 Rainfall

Processed rainfall records from 1978 to 2015 were provided by SPU at 17 rain gauges located within the City of Seattle. These records consist of a continuous one-minute time series of rainfall depth, adjusted to Daylight Saving Time. Rain gauge RG09 is located to the north of the basin, on the westernmost side of the Woodland Park Zoo, and provided the rainfall record for both NPDES148 and NPDES174 in the hydrologic model. The locations of rain gauges relative to Basin 174 and basin boundaries are shown in Figure 2-1 below.

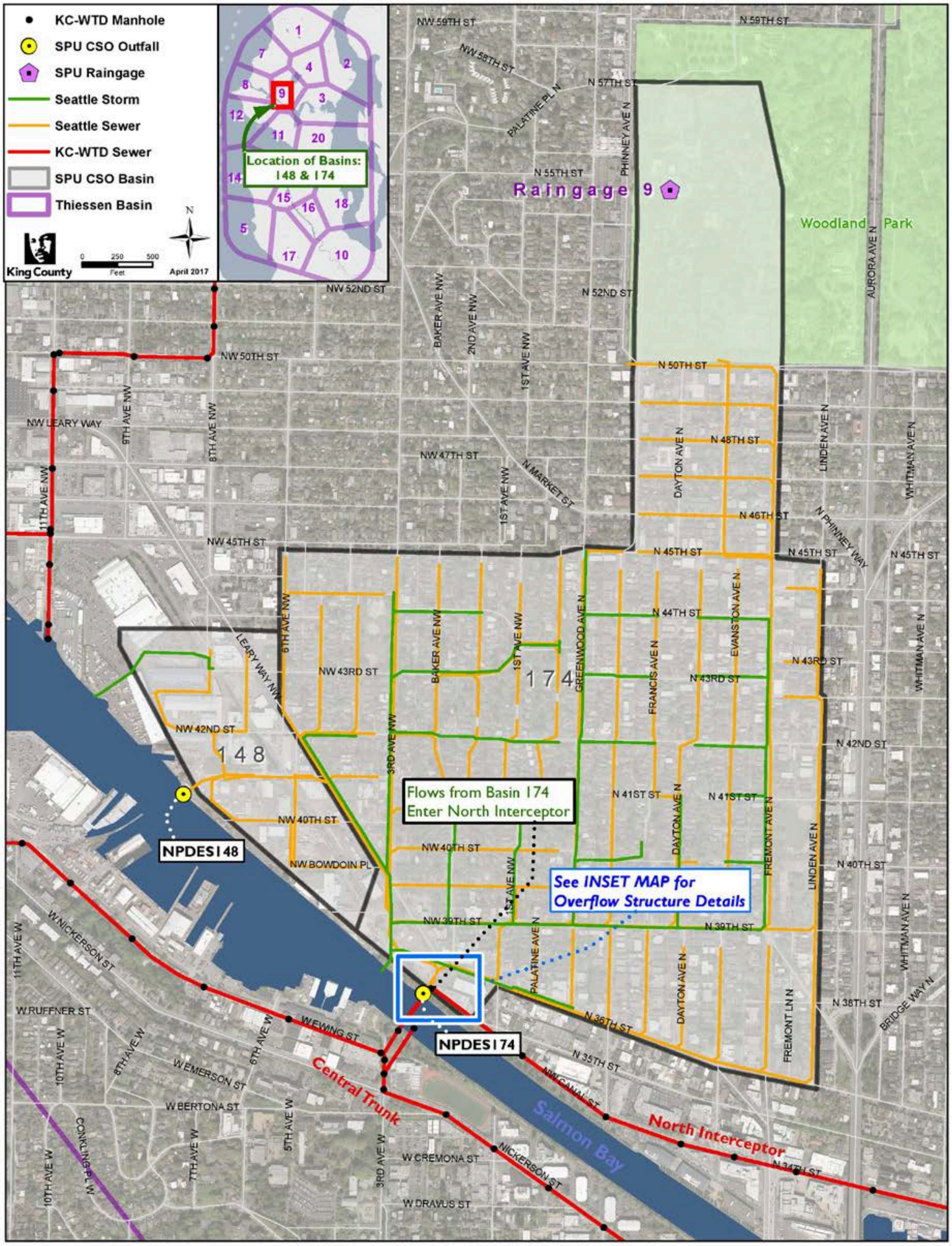

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Figure 2-1: Locations of basin and rain gauge. INSET MAP is Figure 3-1.

2.5 As-Built and Design Engineering Drawings

Archived record drawings of sewer pipes and facilities were available from both WTD and SPU. As-Built and design drawings were used to provide modeling detail for facilities and pipes and to address discrepancies and missing data from GIS shape files.

2.6 SCADA

WTD maintains a historical record of SCADA (Supervisory Control and Data Acquisition) continuously collected from WTD offsite facilities. SCADA data useful for model calibration includes calculated and measured flows, water levels, gate positions, and operational states of pumps and flow regulating structures. SCADA data was used to define a time series for the downstream boundary condition using the interceptor level at the Canal Street Weir. This SCADA measurement is approximately 100’ upstream of the Basin 174 entry point to the North Interceptor. This SCADA data was transposed to an estimated water level in the North Interceptor at the entry point of the Basin 174 basin. Figure 2-2 shows the location of the portable flow meter used for hydrologic calibration, and its relative location in the Fremont area.

2.7 Flow Monitoring

WTD and SPU perform conveyance system monitoring with an array of permanent and portable flow meters. These are generally installed in manholes, and have collected upstream depth and velocities for at least one wet season, from which flow rates can be calculated. Data is typically sampled at either 5 or 15 minute intervals. The data are used for the hydrology model calibration and for the verification of the hydraulic model.

Table 2-A below shows the meters and periods used for the hydrologic calibration, while Figure 2-2 on the following page shows their locations within the basin. Figure 2-3 demonstrates the location of the two flow meters at SPU manhole 021-052, and their locations within the manhole from plan view.

Table 2-A: Flow meter data quality and locations

Flow Meter MH ID	Subsystem	Data Provided	Start Date	End Date	Agency	Address
021-052	MP1 (Dry weather side of weir)	Water Level & Flow	8/1/2014	3/1/2016	SPU	NW 36th St and 2nd Ave NW
021-052	MP2 (Overflow side of weir)	Water Level & Flow	Q: 1/1/2008 WL: 1/1/2008	Q: 2/1/2015 WL: 6/1/2016	SPU	NW 36th St and 2nd Ave NW
021-056	N/A	Water Level	3/6/2009	3/18/2010	SPU	NW Canal St between NW 35th St and NW 36th St
WWPS084	N/A	Wet well level and on/off data	1/1/2008	5/31/2016	SPU	NW Canal St and NW 41st St

For this project, 021-052 (MP1) was used for the hydrologic calibration. The entirety of the upstream basin was included in the hydrologic calibration. 021-052 (MP2) was used to verify the overflow events in the hydraulic verification piece of the model development. 021-056 was a water level data set used for hydraulic verification at the connection to the North Interceptor.

The flow meter data for PS84 was provided by SPU to see the flows coming from NPDES148, upstream of the NPDES174 weir. After exploration of the pump data, an effluent flow from the pump station was developed using on/off data, wet well level, and pump manufacturer information. After reviewing this data, the noisiness of the data proved to be too much, and the magnitude of the flows was not significant enough to the overall 174 basin to warrant further calibration.

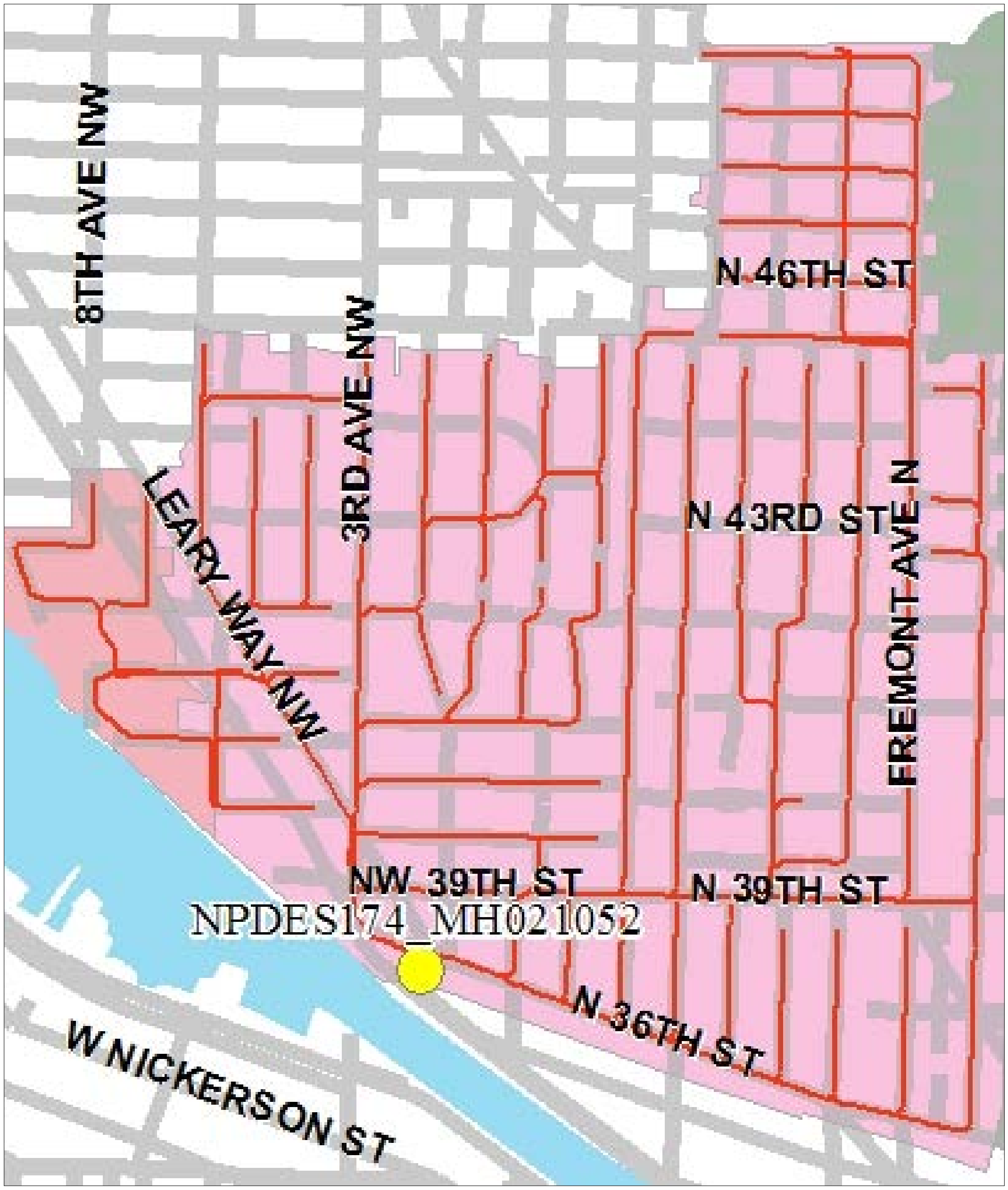


Figure 2-2. Approximate location of 021-052 Flow Meter in Basin 174 Basin

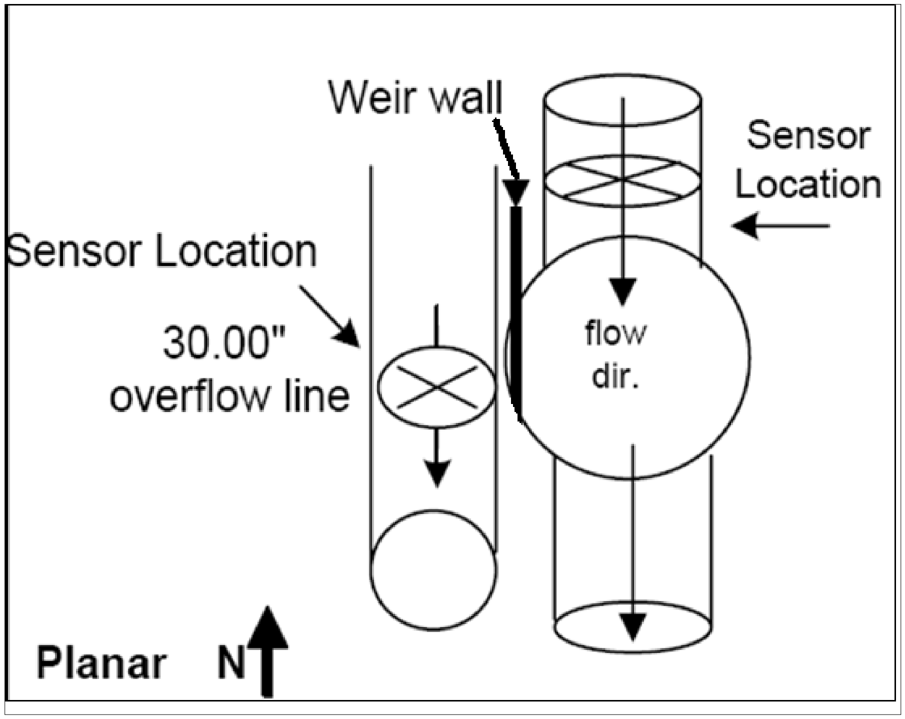


Figure 2-3. Location of sensors in Manhole 021-052. The sensor on the left in this illustration is MP2 (overflow side of weir), and sensor on the right is MP1 (dry weather side of weir)

3 Model Development

The Danish Hydraulics Institute (DHI) MIKE URBAN model was selected to perform hydrologic and hydraulic modeling of the model. The model consists of a hydrologic component and hydraulic component.

Model development consisted first of compiling the different elements of the model (basins, manholes, pipes, and control structures, which, in the case of this particular basin, included the two overflow weirs), connecting them, and assigning them properties consistent with their physical attributes. Second, rainfall, evapotranspiration, dry weather flows (DWF), and Ship Canal and North Interceptor water levels were used to define the boundary conditions for the network. Third, the hydraulics of the conveyance were adjusted, including the physical characteristics of the weir and the head losses at nodes, as the default parameters in MIKE URBAN tend to compute losses that are higher than monitoring data would suggest are realistic.

Figure 3-1 below highlights the modeled conveyance piping, the control structures, the overflow points, and the connection to the North Interceptor at the southernmost portion of the model.

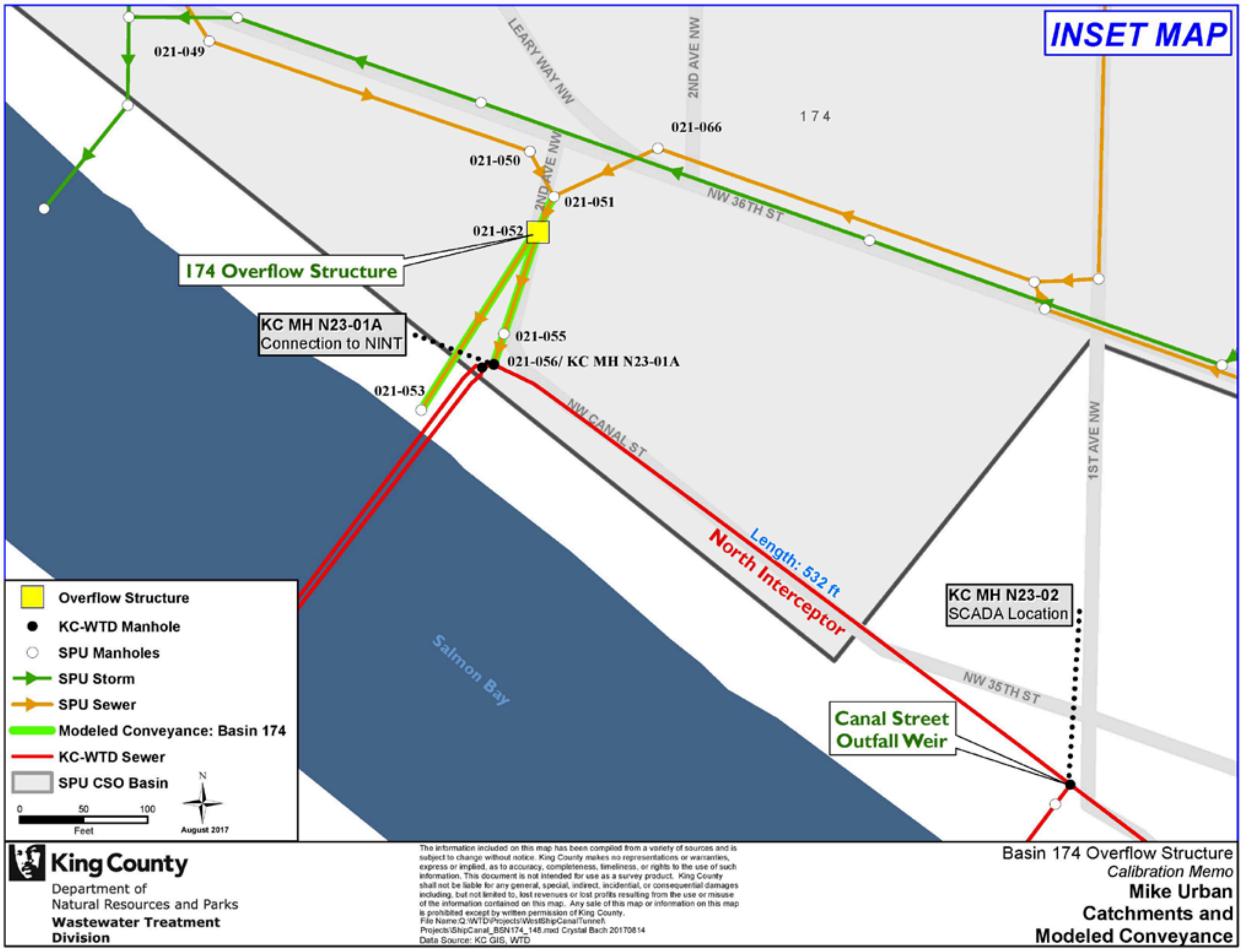


Figure 3-1: Modeled catchments of Basin 174

3.1 Hydrologic Model

The hydrologic component (MOUSE RDII [Rainfall Dependent Infiltration and Inflow] and MOUSE RD Runoff Computation) characterizes the basin response to rainfall in terms of a hydrograph at a tributary location. The hydrograph consists of both surface runoff and RDII. Surface runoff routing is calculated using the Kinematic Wave model (Model B), and accounts for runoff produced from impervious areas of the basin. It's often referred to as the fast response component (FRC). The RDII module accounts for overland flow, interflow, and groundwater processes related to pervious areas of the basin. It's often referred to as the slow response component (SRC). RDII considers evapotranspiration, soil moisture, and groundwater table depth in its calculations.

The process of delineating each sub-basin began by using the previously-defined basins for WTD's *Runoff-Transport* model of the Fremont area, which are based on flow directions throughout the network, as well as the NPDES permitted basins. The 2 sub-basins upstream of the Basin 174 Structure were calibrated together using the flowmeter at the weir. The drainage area of each basin, as well as the relevant flow meter, is given in Table 3-A below.

3.1.1 Summary of Sub-Basins

Table 3-A below summarizes the sub-basins calibrated along with their respective areas, associated rain gauge, and flow meter used for calibrating the sub-basin.

Table 3-A: List of Calibrated Model Basins

Sub-Basin	Basin Area (acres)	Rain Gauge	Flow Meter
NPDES148	38.97	RG09	021-052 MP1 (SPU)
NPDES174	348.96	RG09	021-052 MP1 (SPU)

3.1.1.1 NPDES148

NPDES148 is a small basin to the west of NPDES174, partially separated and connected to NPDES174 by a pump station operated by SPU. With an average DWF of .0595 MGD, this basin provides a very small percentage of the overall contributing flow to the basin. As a result of its location upstream, flow meter data from MH 011-242 was used to validate the overall performance of the basin as a whole, but was not suitable to calibrate the sub-basin individually. The parameters for both NPDES148 and NPDES174 were determined based on the combined response in the same calibration.

3.1.1.2 NPDES174

For basin delineation of Basin 174, there were multiple GIS layers, scenarios and outcomes to consider. The ultimate decision was made to work off of a Runoff Transport delineated basin. This was determined because of the characteristics of the Woodland Park Zoo, both apparent in GIS and the flow meter data downstream. There is evidence of an irregular flow intake in the diurnal pattern in this basin, and in the previous report provided by the City of Seattle indicated that this increase came from the Woodland Park Zoo. This would indicate that the zoo's sanitary sewers are connected to this basin.

Upon looking into the basin's upstream pipes, there is one pipe that is 18", when all the rest of the local pipes are 8". While GIS does not have the local Woodland Park Zoo sanitary lines available, this gives evidence to the input location of the zoo's sewer flows. With this information and the contour information listed in Section 2.2, the current basin boundary was developed.

3.1.2 Summary of Hydrologic Parameters

The FRC requires length and slope parameters for each basin. Length was set to a constant 300 feet for both basins, while slope was set to the average slope across the basin, to the nearest percent. This was determined to be 10%. The length is representative of the average length runoff follows before entering the CSS. The Manning roughness obtained during the hydrologic calibration will adjust as a reflection of any deviation in the slope and length in each basin from the constant values set in the model. The parameters chosen to be calibrated are provided in Table 3-B, and the rest of the hydrologic parameters are set to MIKE URBAN default values.

Table 3-B: Parameters calibrated for the hydrologic model

Parameter Name	Model ID	Description	Units
Impervious Area – Flat	B_A_IFLAT	Fraction of basin area that is impervious and connected to the CSS	%
Impervious Manning Number - Flat	B_M_IFLAT	Defines the roughness of the impervious basin area, used in the hydraulic routing of the runoff (Manning's equation)	N/A
RDII Area	RDII_AREA	Fraction of basin that contributes RDII to the CSS	%
Surface Storage	UMAX	Defines the maximal water contents in surface storage	In
Root Zone Storage	LMAX	Defines the maximal water contents in root zone storage	In
Overland Coefficient	CQOF	Determines the extent to which excess rainfall (after surface storage is retained) runs off as overland flow	N/A
Time Constant	CK	Determines how fast the flow responds to rainfall. Also has some effect on the routing of interflow.	Hr
Time Constant Interflow	CKIF	Together with Umax determines the amount of interflow	Hr
Time Constant Baseflow	CKBF	Determines the hydrograph recession during dry periods	Hr

In addition to these parameters *I_U* and *I_L* (the initial abstractions for surface storage and root zone storage) were fixed at the initial value of *UMAX* and 75% of the initial value of *LMAX*, respectively. The variable *GW_CAREA*, the proportion of the groundwater area to the basin area, was also originally used during PEST calibration, but was later fixed at 100% to ensure model stability.

3.2 Hydraulic Model

The hydraulic component (MOUSE HD Pipe Flow Computation) further characterizes the hydrologic response to rainfall in terms of flow and water levels within the CSS. MOUSE HD Pipe Flow uses a Dynamic Wave model to route flow within a network of nodes and links. Nodes represent manholes or outlets; links represent pipes, storage tanks and control devices such as pumps, valves, weirs and regulated gates. The hydraulic network used to represent the Basin 174 system is highlighted in green in Figure 3-1. Section 3.4 describes the control structures in further detail. Most parts of the hydraulic model were not calibrated per se; instead, existing dimensions, facility operation manuals, engineering drawings, and monitoring data were used to match the model’s hydraulic parameters to physical observations, and adjustments were made where necessary to ensure model stability and realistic results. The only exception is with the overflow structures, where a variety of approaches were used in an attempt to match simulated overflow data to observed overflow data. This verification is discussed in detail in Section 5.

3.3 Boundary Conditions

The boundary conditions, or loading inputs, for the hydrology are rainfall and evapotranspiration. These have been described in Section 2. The boundary conditions for the hydraulic model are the dry weather flow (DWF) and downstream water levels, both in Lake Washington Ship Canal and North Interceptor.

3.3.1 Dry Weather Flows

The DWF is the combination of groundwater infiltration and sanitary flow regularly entering the collection system regardless of precipitation. Meter data during dry weather periods were used to create a set of dry weather diurnal curves for all basins. Three diurnal curves were created for each basin to reflect average observed flows during weekdays, Saturdays, and Sundays. The 5-minute flow meter data was averaged into hourly flow rates to get the 3 DWF patterns for each sub-basin.

Dry weather flows were developed using the 021-052 flow data. 3 different periods were used: 08/22/2014 – 08/30/2014, 06/11/2015 – 06/19/2015, and 07/11/2015 – 07/24/2015. The 3 different periods were used due to several factors. These include the lack of continuous flow data for a long enough sample to provide an adequate sample size, and the irregular flows from the Woodland Park Zoo which become more normalized with more date ranges. The woodland park flows were included in the averaging of the dry weather flow for these date ranges, to account for the volume of water contributed from these basins on an average day.

3.3.2 Downstream Water Levels

Water levels were used as the downstream boundary conditions to both the overflow side of the model and the part of the model replicating flows to the North Interceptor. SCADA data for the Ship Canal water level was used as a downstream boundary condition from 2005 - 2016. For the long term simulations spanning back to 1978, an UNSTDY model run of the downstream boundary condition was used as the downstream water level from 1978-2005.

At the outlet of the model into the North Interceptor, SCADA data was used to develop a time series of the water level in the inlet trunk. This water level is measured approximately 530’ upstream of the Basin 174 inlet point to the North Interceptor. Additional modeling using UNSTDY was necessary to extend the timeseries and cover the entirety of the long term simulation period.

3.4 Pumps and Control Structures

This model includes two control structures: the overflow weir at MH 021-052, referred to in this report as NPDES174 overflow weir, and the flap gate at 021-056.

3.4.1 NPDES174 Overflow Weir

Mike Urban models weirs without upper bounds, which can present challenges in sewer systems with ceilings. Therefore, the weir at 021-052 was modeled as an orifice to simulate the top of the weir chamber, providing more realistic conditions. In addition, the orifice calculations in Mike Urban provide computations for 4 different flow regimes of an orifice, which is preferred over the weir calculation in Mike Urban.

3.4.2 NPDES174 Flap Gate

Basin 174 has a flap gate at the entry point of the basins' flow into the North Interceptor. This was modeled as a one-way circular orifice in the model.

3.5 Losses in Nodes

Head losses in nodes due to manholes and junctions are computed in Mike Urban. The standard calculation usually overestimates the losses, and thus custom losses were set for the nodes in the model. The custom loss estimates follow the Weighted Inlet Energy method (also referred to as Mean Energy Approach), with a coefficient type of total head loss (Total HLC) applied at the outlet of the node. Five standard cases were used in this model. The loss coefficients used are listed in Table 3-C.

Table 3-C: Head Loss Coefficients

Type of Node	Loss Coefficient
45 Bend	.1
90 Bend	.2
Channelized	.05
Junction	No head loss
Outlet	1

The node type was set to "Junction" to manually force the head loss through the node to be zero.

3.6 Real-Time Control (RTC)

RTC was not used due to the lack of any movable control structures in the basins.

4 Hydrologic Calibration

Model calibration is the process of iteratively adjusting model parameters until the results of the model most closely approximate real world observations. This is done by adjusting the hydrologic parameters of the model for each calibrated basin shown in Table A-2. This iterative process has been automated using Model-Independent Parameter Estimation and Uncertainty Analysis (PEST) software. PEST employs a Gauss-Marquardt-Levenberg method of steepest descent to minimize the differences between model results and observations. Sometimes after the automatic calibration, a manual adjustment of the parameters is carried out to improve the match between model and observations.

Prior to calibration, events are first identified from the monitoring data and assigned appropriate weighting factors. Events with snow are avoided, as it is difficult to model melt rates accurately. Likewise, strong wind events can lead to rainfall measuring errors and should also be avoided. During calibration, the model is run for a sufficient duration to encompass the calibration events. Following calibration, the goodness-of-fit of the results are evaluated using several statistical criterion and metrics. A spin-up period of at least 2 wet seasons prior to the first calibration event is recommended for the hydrologic calibration.

4.1 Events

When possible, suitable events for calibration were manually selected by reviewing the meter time series. Ten flow events of varying magnitude and duration were identified for each. Initially, the basin's flow meter had a stark change of measurement on 07/13/2015. See Figure 4-1 for the differences in flows in this time period. Because of this, the period after the change in flow pattern was used, assuming that the change in flow pattern was based on verification from SPU. The original calibration only used events from 07/14/2015 – 8/1/2016.

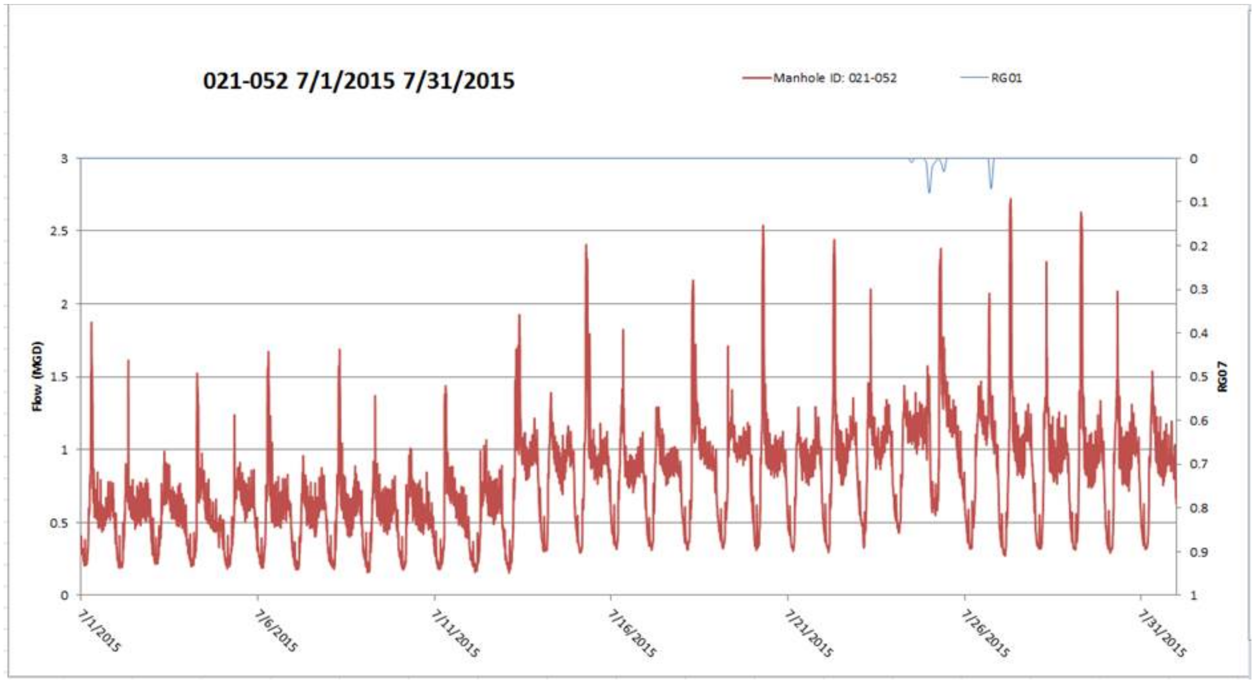


Figure 4-1. Change in flow pattern to flow meter 021-052

After being provided feedback from the city, it became clear that the data originally used for calibration had a change in flow data due to an error in the translation of the data. After revised data was provided, new storms over a longer duration were selected for the calibration of Basin 174. The final storms used for hydrologic calibration included 1 event purely for dry weather flow and 9 events of varied intensity and duration. The final calibration period extended from August, 2015 to February, 2017. The periods used in the calibration are evident in the calibration plots in section 6.

4.2 PEST Weighting

PEST has the ability to weight individual observations during the calibration. PEST will dedicate more computational effort to matching observations with weights above 1, and less effort for weights below 1. Observations with zero weight are effectively ignored.

The selection of an appropriate weighting strategy is dependent largely upon the nature of the calibration and PEST's performance without a weighting scheme. For this particular model, the calibration performed adequately without the use of a variable weighting scheme. All valid values were assigned a weight of 1, and missing or clearly inaccurate data were assigned a weight of 0 to discount their contribution from the calibration error measurement and decision process. There were 2 portions of the events that were consistently given a weight of 0:

- 12/08/2015 10:45 PM – 12/09/2015 5:35 AM
- 01/21/2016 8:30 AM – 01/22/2016 1:15 AM

Both periods saw significant error or lack of values in the flow meter, and both were during large events that would cause large error in the PEST calibration.

4.3 Manual Calibration

After the PEST calibration, an analysis of the resulting RDII flow components showed which of those were under- or over-estimated. Adjustment of the parameters based on this information improved the model fit to data for most basins. In general, manual calibration was used to adjust the hydrologic parameter *B_A_Iflat* and was used to attain a more desirable fit for the peak flow values in cases where the model consistently under- or over-estimated the peak flow values. For the final calibration, the *B_A_Iflat* parameter was increased from the PEST value of 4.79 to the final calibrated value of 7.00.

4.4 Evaluation of Results

The statistical criteria used to evaluate the goodness-of-fit between measured observations and calibrated model predictions for this modeling effort are the standardized Bias, Nash-Sutcliffe model efficiency coefficient, the Root-Mean-Square Error (RMSE), the Relative Peak Flow Difference and Relative Total Volume Difference.

4.4.1 Bias

Bias indicates a general shift of the models, and ranges from positive to negative infinity, where 0.0 indicates a perfect fit. Positive and negative bias indicates model overestimation and underestimation, respectively. Bias is expressed as:

$$B_s = \frac{\sum_{i=1}^{m}\left(Q_i^{mdl} - Q_i^{obs}\right)}{m * \overline{Q^{obs}}}$$

where m is the number of values in the calibration event, Q_i^{mdl} is a model value, Q_i^{obs} is an observed value, and $\overline{Q^{obs}}$ is the average observed value.

4.4.2 Nash-Sutcliffe Coefficient

The Nash coefficient is one less the ratio of the sum of the squared differences between modeled and observed values and the sum of the squared differences between the observed and mean observed values. Nash values can range from 1.0 to negative infinity, where 1.0 indicates a perfect fit. In practice, the Nash coefficient served as the primary goodness-of-fit indicator. Values of 0.9 and greater indicated excellent agreement between the observations and the model. The Nash coefficient is expressed as:

$$Nash = 1 - \frac{\sum_{i=1}^{m}(Q_i^{mdl} - Q_i^{obs})^2}{\sum_{i=1}^{m}(Q_i^{obs} - \overline{Q}_i^{obs})^2}$$

where m is the number of values in the calibration event, Q_i^{mdl} is a model value, Q_i^{obs} is an observed value, and $\overline{Q}_i^{obs}$ is the average observed value in the event.

4.4.3 Root-Mean-Square Error

RMSE is the square root of the average squared difference between the observed and model values. RMSE values can range from 0.0 to positive infinity, where 0.0 indicates a perfect fit. RMSE is expressed as:

$$RMSE = \sqrt{\frac{1}{m}\sum_{i=1}^{m}(Q_i^{mdl} - Q_i^{obs})^2}$$

where m is the number of values in the calibration event, Q_i^{mdl} is a model value, and Q_i^{obs} is an observed value.

4.4.4 Relative Peak Flow Difference and Relative Total Volume Difference

Two additional metrics further quantified the goodness-of-fit. These compared the relative difference of model peak from observed peak, and total model volume from total observed volume. Values of 0.0 for both peak flow and total volume indicate perfect agreement. Positive or negative values indicate model overestimation or underestimation, respectively. The relative peak flow and relative total volume are expressed as:

$$Q_{pk\_diff} = \frac{Q_{\max}^{mdl} - Q_{\max}^{obs}}{Q_{\max}^{obs}} \; ; \; V_{diff} = \frac{V_{tot}^{mdl} - V_{tot}^{obs}}{V_{tot}^{obs}}$$

where $Q_{\max}^{mdl}$ is the maximum model value and $Q_{\max}^{obs}$ is the maximum observed value, V_{tot}^{mdl} is the total model volume, and V_{tot}^{obs} is the total observed volume.

4.5 Hydrologic Parameters

The hydrologic parameters calibrated in the model were discussed in Table 3-B. Calibrated values for these parameters are tabulated by basin in Section 6.

5 Hydraulic Verification

In order to accurately model the frequency and volume of CSO events, the SPU overflow structure at 021-052 required calibration.

5.1 NPDES174 Overflow Weir

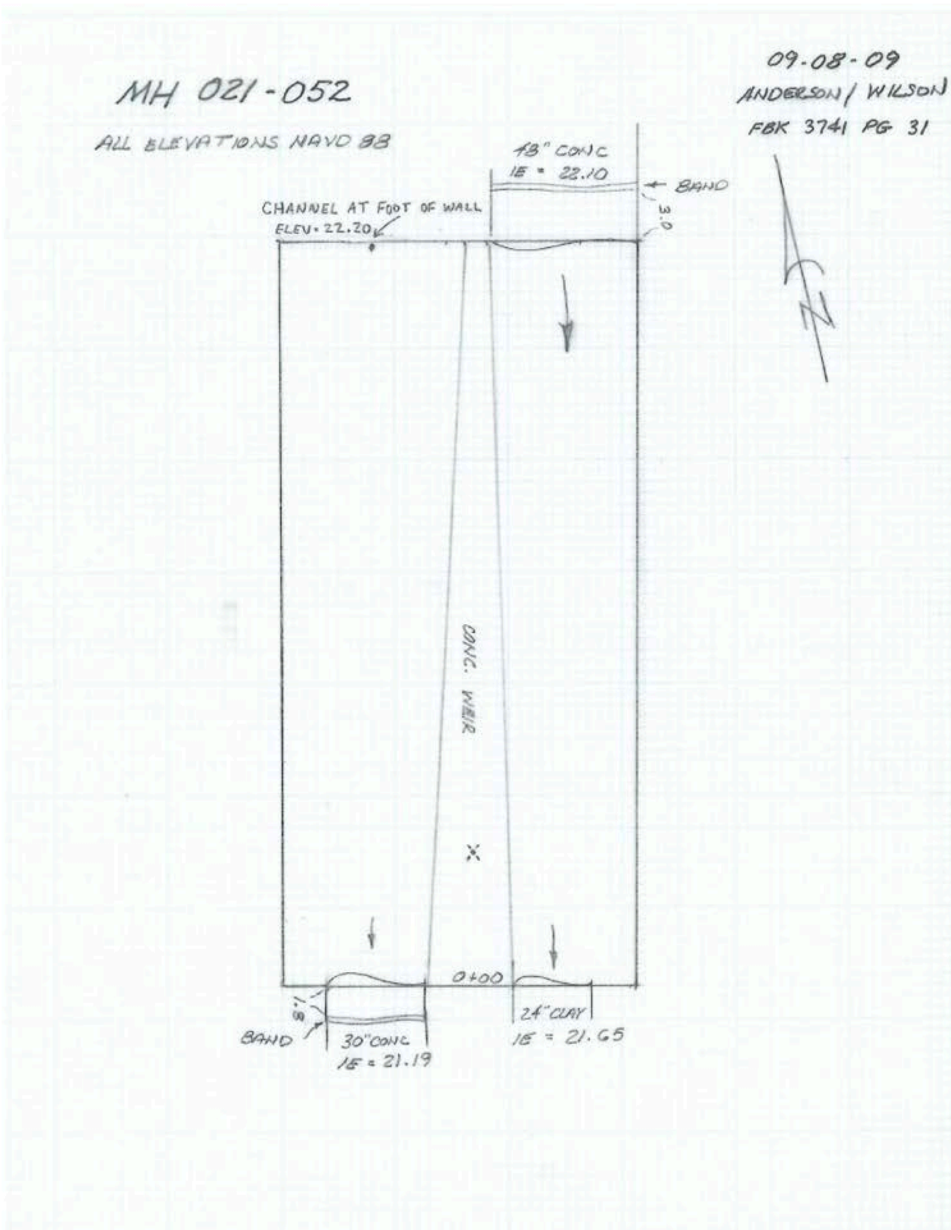


Figure 5-1: Engineering drawing for the MH021-052 Overflow Weir.

The overflow structure at MH021-052 is a transverse weir, but behaves similar to a perpendicular broad-crested weir. The flow is backwatered during overflow events, causing the water level to rise in the weir chamber at a sub critical state. The overflow line is a 30” pipe that heads south into the Lake Washington Ship Canal.

The results of the hydraulics of the Basin 174 model were compared to SPU’s reported overflow frequencies and volumes provided from the 021-052 MP2 flow meter data. After 02/2015, when the MP2 flow meter data was no longer calculated, a weir equation using water depth above the weir was used as the measured overflow.

Due to the constraints of accurately representing the overflow weir in MIKE URBAN, variations of the real-life weir were attempted in the model. Weir crest elevation, Weir coefficient and width were modified, ultimately to come back to similar values to real life conditions. The following table outlines the parameters used for the NPDES174 CSO Weir.

Table 5-A: MIKE URBAN Orifice parameters for NPDES174 CSO Weir

Control Structure Type:	Rectangular Orifice	
Oper. Mode:	No Control	
Weir Crest Elevation:	120.33	ft
Flap:	FALSE	
Discharge coefficient:	1.00	
Height from weir crest to vault ceiling:	3.75	ft
Width of weir:	20.00	ft

The performance of the modeled overflow structure was compared to observed flow meter data collected by SPU and compiled in the Annual CSO Status Reports. Overall, the overflow volumes matched up well between model and measured values. Please see Table 5-B and Figure 5-2 for the results for the hydraulic verification of Basin 174.

Table 5-B: Basin 174 Overflow Comparison

NPDES174 Overflow Weir Comparison						
Year	Modeled Overflow Frequency	SPU Reported Overflow Frequency	Percent Error	Modeled Overflow Volume (MG)	SPU Reported Overflow Volume (MG)	Percent Error
2006	20	22	-9%	11.452	14.962	-23.5%
2007	10	10	0%	18.015	23.489	-23.3%
2008	8	7	14%	2.533	0.903	180.6%
2009	19	16	19%	14.626	8.914	64.1%
2010	21	19	11%	17.866	13.179	35.6%
2011	14	16	-13%	13.375	8.548	56.5%

2012	20	20	0%	22.907	18.486	23.9%
2013	12	12	0%	6.785	6.399	6.0%
2014	24	25	-4%	10.214	11.174	-8.6%
2015	18	17	6%	9.160	9.163	0.0%
Average:	16.6	16.4	1%	12.693	11.522	10%

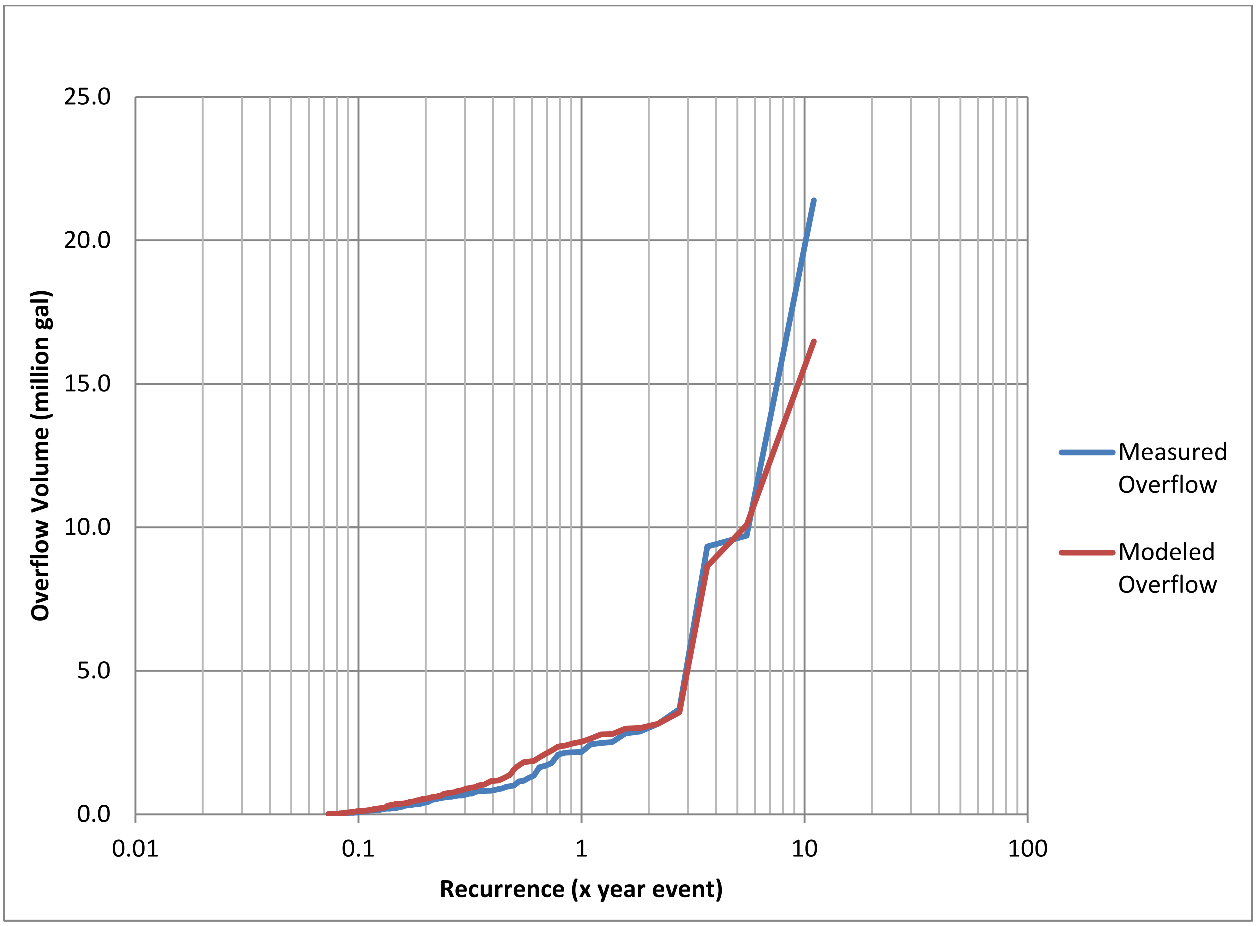


Figure 5-2: Recurrence of Basin 174 overflow volumes (2006 – 2015)

6 Results

6.1 Hydrologic Calibration

The calibrated parameters for each sub-basin are given in Table 5-A. For the calibrated basins, the evaluation statistics are presented in Tables 5-B through 5-H. Graphs showing the calibrated model output, observed flow, and precipitation for each sub-basin in included in Appendix A.

Table 6-A Calibrated Parameters and Goodness-of-Fit Metrics for Basin 174

Parameters	NPDES148 & NPDES174
RDII_AREA	24.9 %
B_A_IFlat	7.0 %
B_M_IFlat	.257
UMAX	3.31 in
LMAX	3.57 in
CQOF	.23
CK	12.1 hr
CKIF	983.8 hr
CKBF	2114.9 hr

NPDES148 & NPDES174					
Event	Bias	RMSE	Nash	Q_{pk} Error	V_{tot} Error
01	21.7%	0.28	0.62	6.0%	22.7%
02 (DWF)	5.6%	0.18	0.42	-19.9%	2.7%
03	13.4%	0.45	0.18	28.6%	38.5%
04	8.1%	0.45	0.34	52.7%	12.9%
05	5.1%	0.32	0.56	33.4%	23.5%
06	5.1%	0.22	0.77	-18.5%	1.5%
07	11.7%	0.35	0.71	-0.3%	17.6%
08	5.0%	0.43	0.66	4.7%	0.4%
09	2.9%	0.24	0.78	-9.9%	3.3%
10	3.4%	0.38	0.59	10.1%	8.0%
Average:	7.6%	0.34	0.72	52.7%	11.5%

6.1.1 Calibration Graphs

Please see appendix A for the hydrologic calibration graphs for Basin 174. These graphs show the simulated flow, flow meter, and rain gauge time series used to calibrate the sub-basins, as well as the calibrated hydrologic parameters and various goodness-of-fit metrics.

6.2 Long Term Statistics

In order to determine relevant statistics on the model's ability to accurately simulate overflows, the model was run for the entire period of available rainfall data (1978-2016). The first year of rainfall data (1978) was repeated for two years prior in order to establish antecedent soil moisture conditions, and the hydrologic engine was run starting in 1976 using this augmented rainfall timeseries.

The one-year overflow volume for the entire 38-year period of record (the 38th event when ranking all events by volume) is 1.290 million gallons. Table 6-B below shows the statistics for each 20-year window in the period of record. The maximum 1-year overflow volume occurs in the 1996-2015 date range.

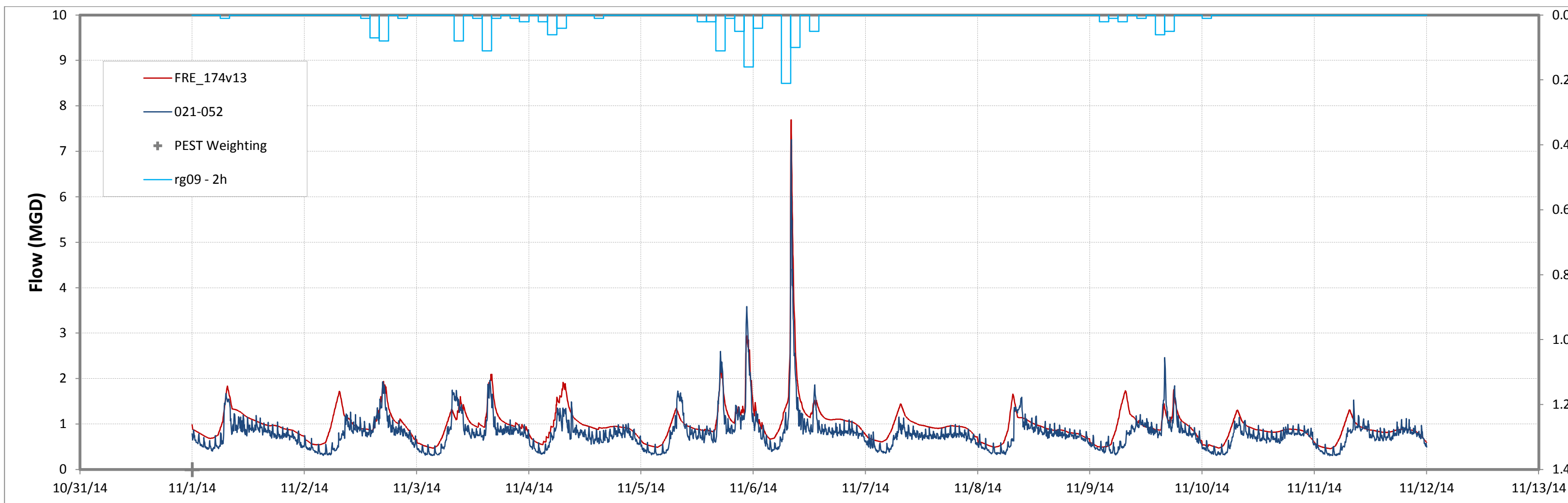
Table 6-B. Basin 174 20-year Rolling Average of Overflow events

Start Year	End Year	# of events in range:	Events/year	Volume/year (MG)	1 yr Overflow Volume	# of events above 1 year volume of 1.29 MG
1978	1997	199	9.95	4.16	1.107	19
1979	1998	194	9.7	4.16	1.084	19
1980	1999	191	9.55	3.95	1.069	18
1981	2000	175	8.75	3.66	0.955	16
1982	2001	168	8.4	3.45	0.955	16
1983	2002	159	7.95	3.31	1.047	16
1984	2003	148	7.4	3.05	0.955	15
1985	2004	139	6.95	3.06	0.955	15
1986	2005	139	6.95	3.10	0.955	15
1987	2006	149	7.45	3.21	1.047	16
1988	2007	142	7.1	3.39	0.955	15
1989	2008	141	7.05	3.34	0.955	15
1990	2009	153	7.65	3.62	1.047	14
1991	2010	155	7.75	3.82	1.011	13
1992	2011	162	8.1	4.10	1.054	15
1993	2012	175	8.75	4.61	1.150	19
1994	2013	179	8.95	4.76	1.173	19
1995	2014	194	9.7	5.23	1.259	20
1996	2015	204	10.2	5.67	1.348	23
1997	2016	194	9.7	5.38	1.290	20

See appendix B for the hydraulic verification information, including water level verification graphs at 021-052 and the recurrence interval chart for NPDES174.

Appendix A: Hydrologic Calibrations

Fremont Basin 174 Model Development and Calibration Report

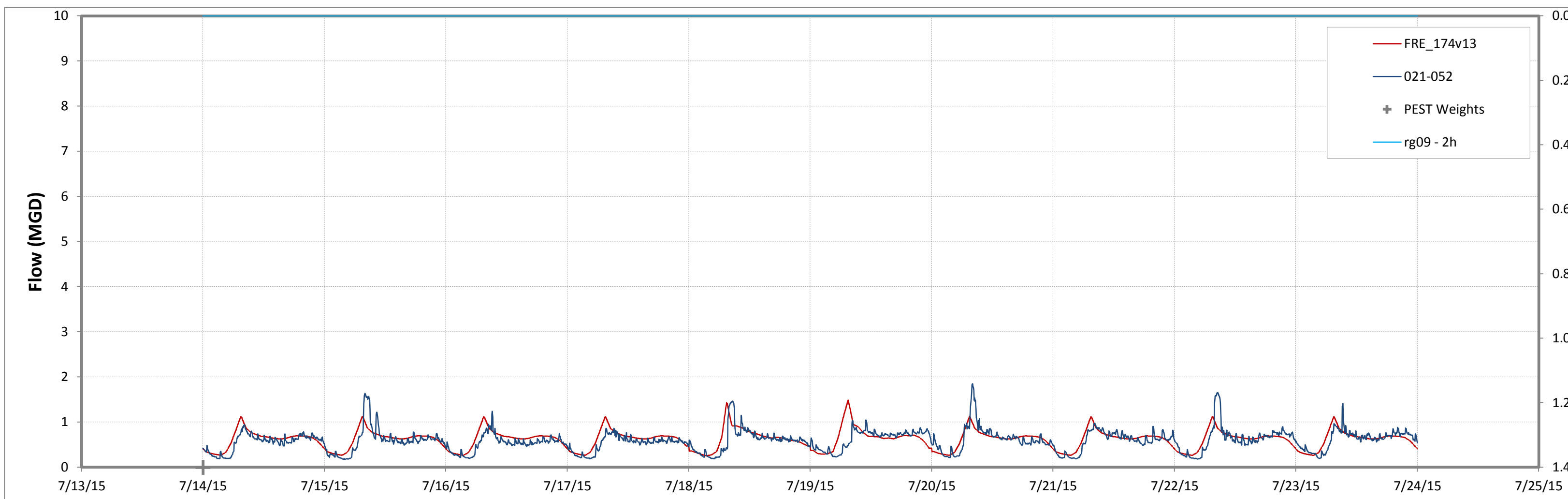


Event 01

11/01/14 to 11/12/14

PEST Optimization	
RDII_AREA	24.9 %
B_A_IFLAT	7.0 %
B_M_IFLAT	0.257
UMAX	3.31 in
LMAX	3.57 in
CQOF	0.23
CK	12.1 hr
CKIF	983.8 hr
CKBF	2114.9 hr

Goodness-of-Fit	
Threshold	0.00
Bias	21.73%
RMSE	0.28
Nash	0.62
Q_{pk} Error	6.03%
V_{tot} Error	22.65%

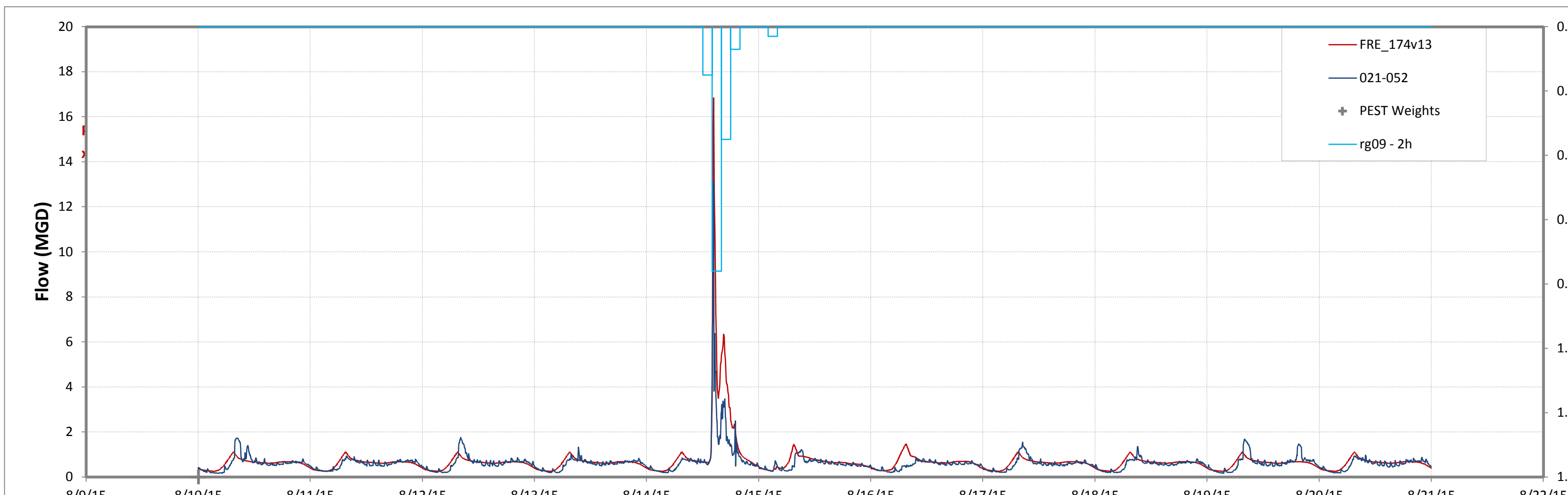


Event 02

07/14/15 to 07/24/15

PEST Optimization	
RDII_AREA	24.9 %
B_A_IFLAT	7.0 %
B_M_IFLAT	0.257
UMAX	3.31 in
LMAX	3.57 in
CQOF	0.23
CK	12.1 hr
CKIF	983.8 hr
CKBF	2114.9 hr

Goodness-of-Fit	
Threshold	0.00
Bias	5.64%
RMSE	0.18
Nash	0.42
Q_{pk} Error	-19.95%
V_{tot} Error	2.67%

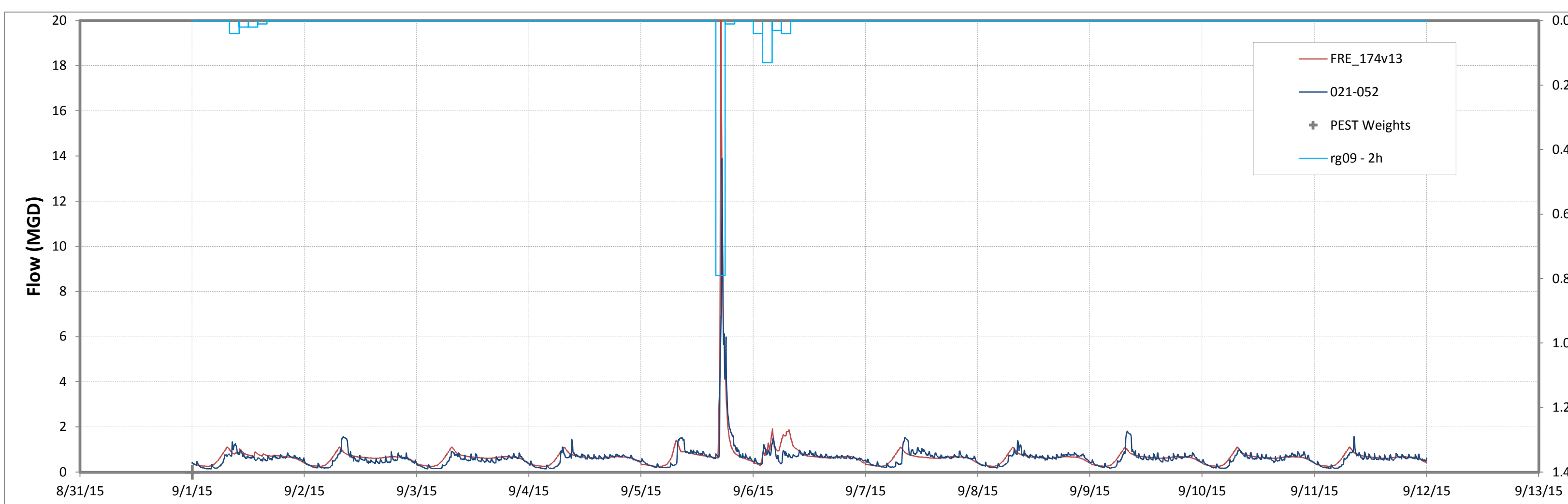


Event 03

08/10/15 to 08/21/15

PEST Optimization	
RDII_AREA	24.9 %
B_A_IFLAT	7.0 %
B_M_IFLAT	0.257
UMAX	3.31 in
LMAX	3.57 in
CQOF	0.23
CK	12.1 hr
CKIF	983.8 hr
CKBF	2114.9 hr

Goodness-of-Fit	
Threshold	0.00
Bias	13.45%
RMSE	0.45
Nash	0.18
Q_{pk} Error	28.59%
V_{tot} Error	38.50%

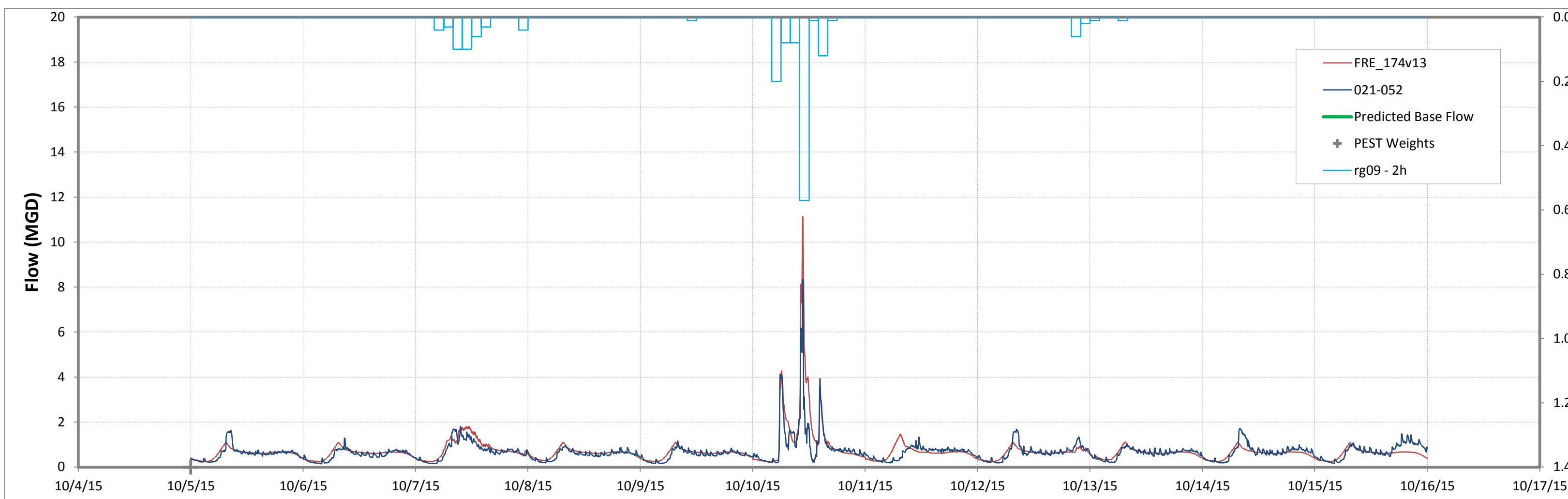


Event 04

09/01/15 to 09/12/15

PEST Optimization	
RDII_AREA	24.9 %
B_A_IFLAT	7.0 %
B_M_IFLAT	0.257
UMAX	3.31 in
LMAX	3.57 in
CQOF	0.23
CK	12.1 hr
CKIF	983.8 hr
CKBF	2114.9 hr

Goodness-of-Fit	
Threshold	0.00
Bias	8.08%
RMSE	0.45
Nash	0.34
Q_{pk} Error	52.71%
V_{tot} Error	12.95%

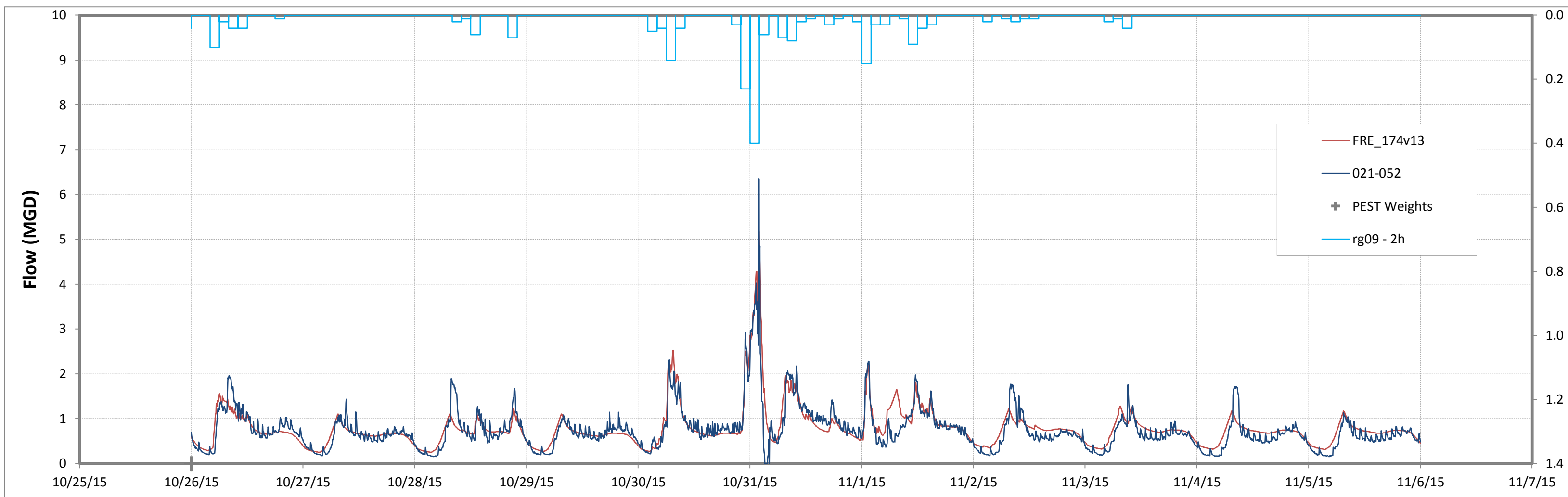


Event 05

10/05/15 to 10/16/15

PEST Optimization	
RDII_AREA	24.9 %
B_A_IFLAT	7.0 %
B_M_IFLAT	0.257
UMAX	3.31 in
LMAX	3.57 in
CQOF	0.23
CK	12.1 hr
CKIF	983.8 hr
CKBF	2114.9 hr

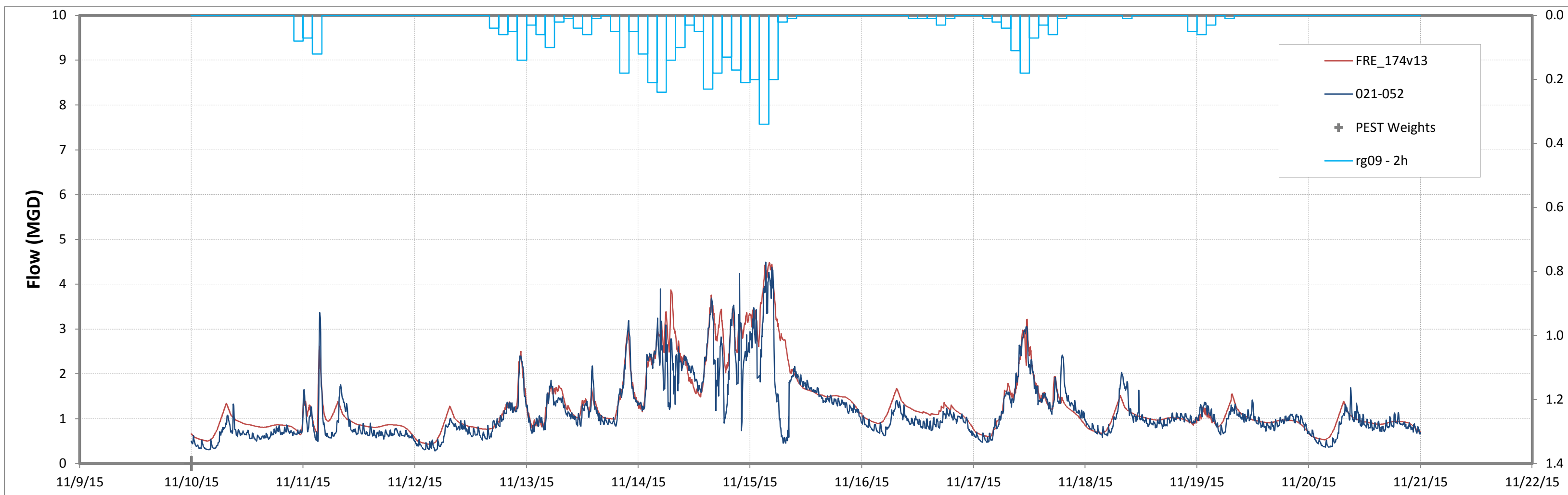
Goodness-of-Fit	
Threshold	0.00
Bias	5.12%
RMSE	0.32
Nash	0.56
Q_{pk} Error	33.39%
V_{tot} Error	23.50%



Event 06
10/26/15 to 11/06/15

PEST Optimization	
RDII_AREA	24.9 %
B_A_IFLAT	7.0 %
B_M_IFLAT	0.257
UMAX	3.31 in
LMAX	3.57 in
CQOF	0.23
CK	12.1 hr
CKIF	983.8 hr
CKBF	2114.9 hr

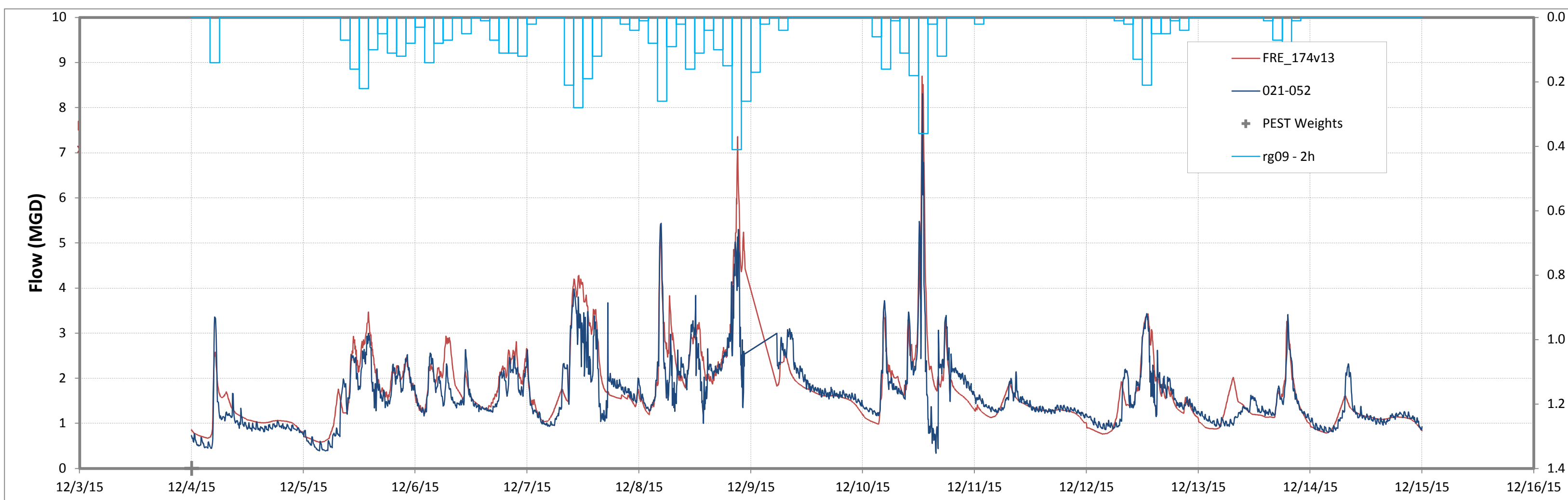
Goodness-of-Fit	
Threshold	0.00
Bias	5.13%
RMSE	0.22
Nash	0.77
Q_{pk} Error	-18.51%
V_{tot} Error	1.49%



Event 07
11/10/15 to 11/21/15

PEST Optimization	
RDII_AREA	24.9 %
B_A_IFLAT	7.0 %
B_M_IFLAT	0.257
UMAX	3.31 in
LMAX	3.57 in
CQOF	0.23
CK	12.1 hr
CKIF	983.8 hr
CKBF	2114.9 hr

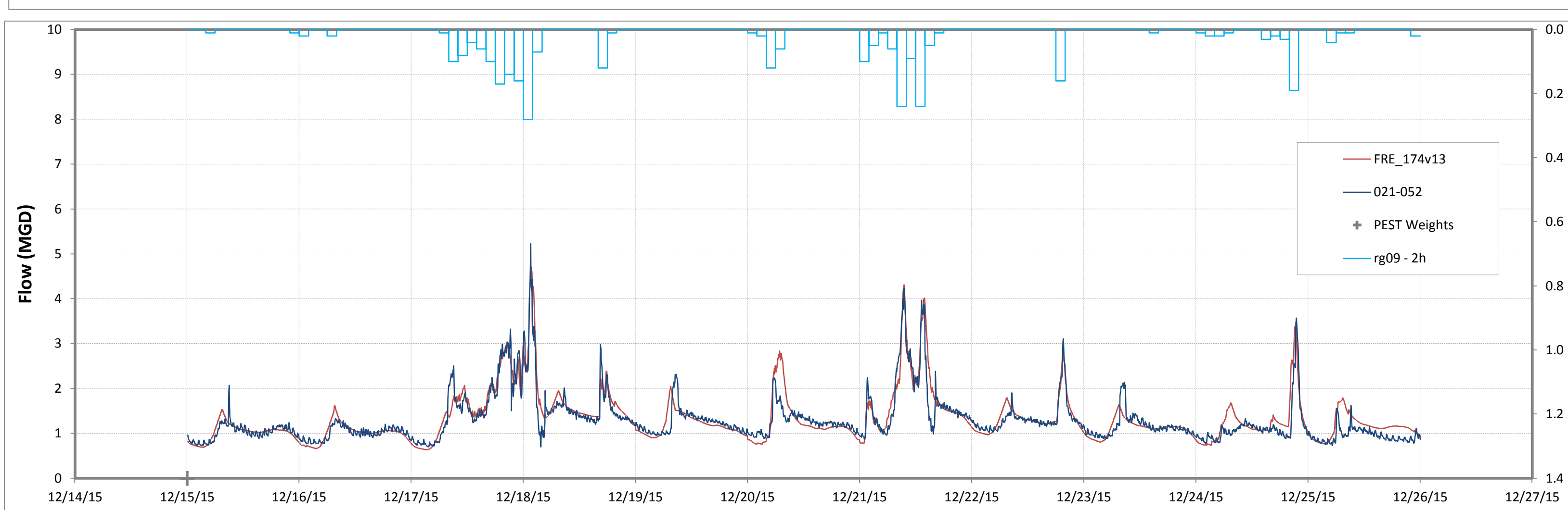
Goodness-of-Fit	
Threshold	0.00
Bias	11.73%
RMSE	0.35
Nash	0.71
Q_{pk} Error	-0.30%
V_{tot} Error	17.62%



Event 08
12/04/15 to 12/15/15

PEST Optimization	
RDII_AREA	24.9 %
B_A_IFLAT	7.0 %
B_M_IFLAT	0.257
UMAX	3.31 in
LMAX	3.57 in
CQOF	0.23
CK	12.1 hr
CKIF	983.8 hr
CKBF	2114.9 hr

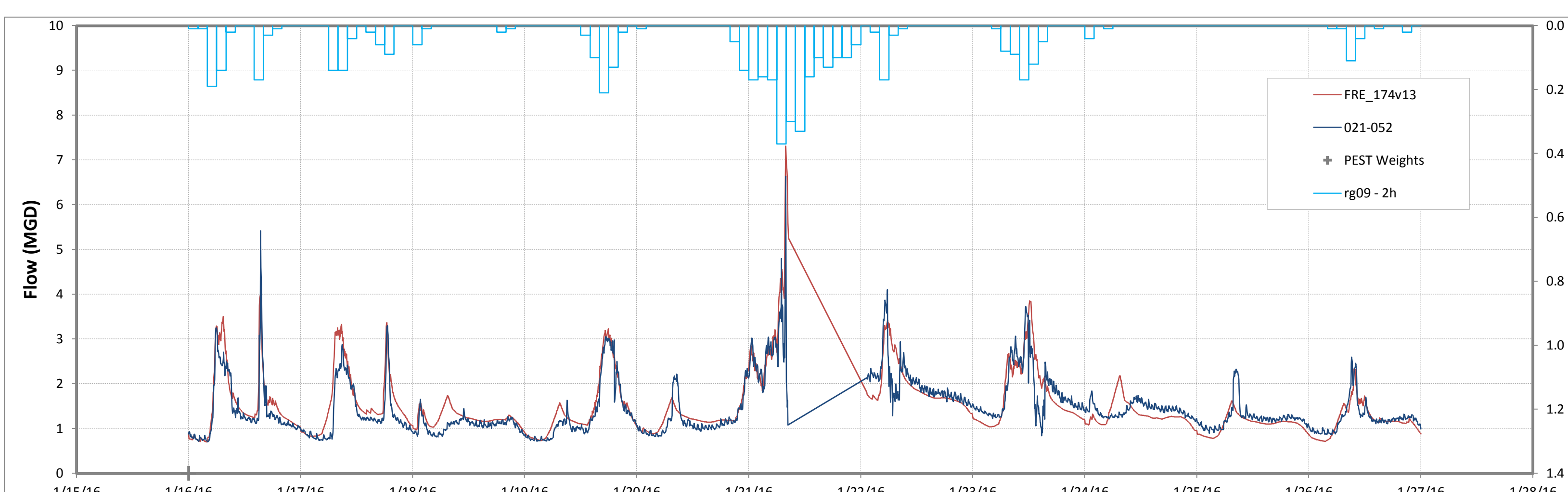
Goodness-of-Fit	
Threshold	0.00
Bias	5.02%
RMSE	0.43
Nash	0.665
Q_{pk} Error	4.69%
V_{tot} Error	0.36%



Event 09
12/15/15 to 12/26/15

PEST Optimization	
RDII_AREA	24.9 %
B_A_IFLAT	7.0 %
B_M_IFLAT	0.257
UMAX	3.31 in
LMAX	3.57 in
CQOF	0.23
CK	12.1 hr
CKIF	983.8 hr
CKBF	2114.9 hr

Goodness-of-Fit	
Threshold	0.00
Bias	2.93%
RMSE	0.24
Nash	0.78
Q_{pk} Error	-9.94%
V_{tot} Error	3.26%



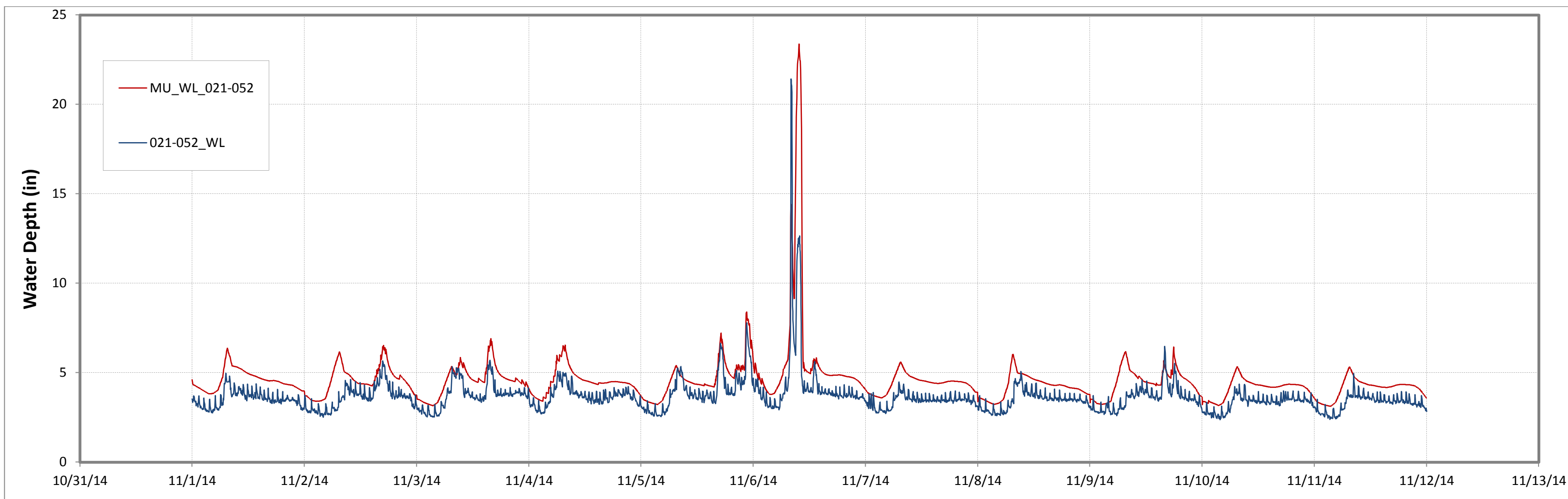
Event 10
01/16/16 to 01/27/16

PEST Optimization	
RDII_AREA	24.9 %
B_A_IFLAT	7.0 %
B_M_IFLAT	0.257
UMAX	3.31 in
LMAX	3.57 in
CQOF	0.23
CK	12.1 hr
CKIF	983.8 hr
CKBF	2114.9 hr

Goodness-of-Fit	
Threshold	0.00
Bias	3.40%
RMSE	0.38
Nash	0.59
Q_{pk} Error	10.12%
V_{tot} Error	8.00%

Appendix B: Hydraulic Verification Graphs

Fremont Basin 174 Model Development and Calibration Report

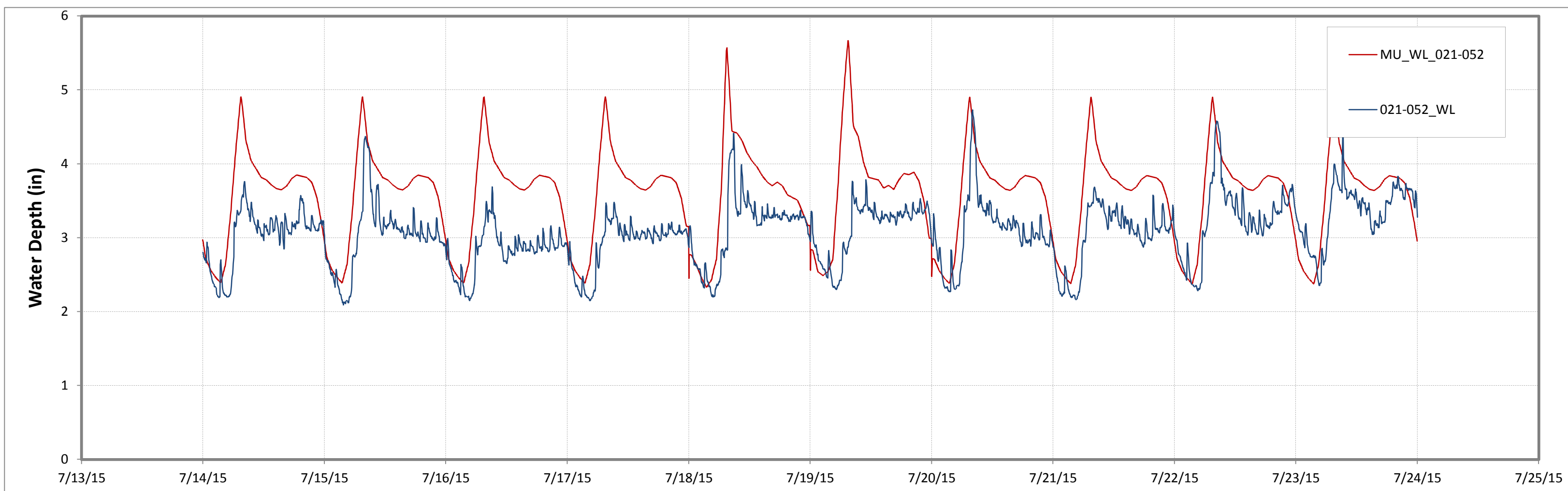


Event 01

11/01/14 to 11/12/14

PEST Optimization	
RDII_AREA	24.9 %
B_A_IFLAT	7.0 %
B_M_IFLAT	0.257
UMAX	3.31 in
LMAX	3.57 in
CQOF	0.23
CK	12.1 hr
CKIF	983.8 hr
CKBF	2114.9 hr

Goodness-of-Fit	
Threshold	0.00
Bias	#N/A
RMSE	#N/A
Nash	#N/A
Peak Error	9.16%

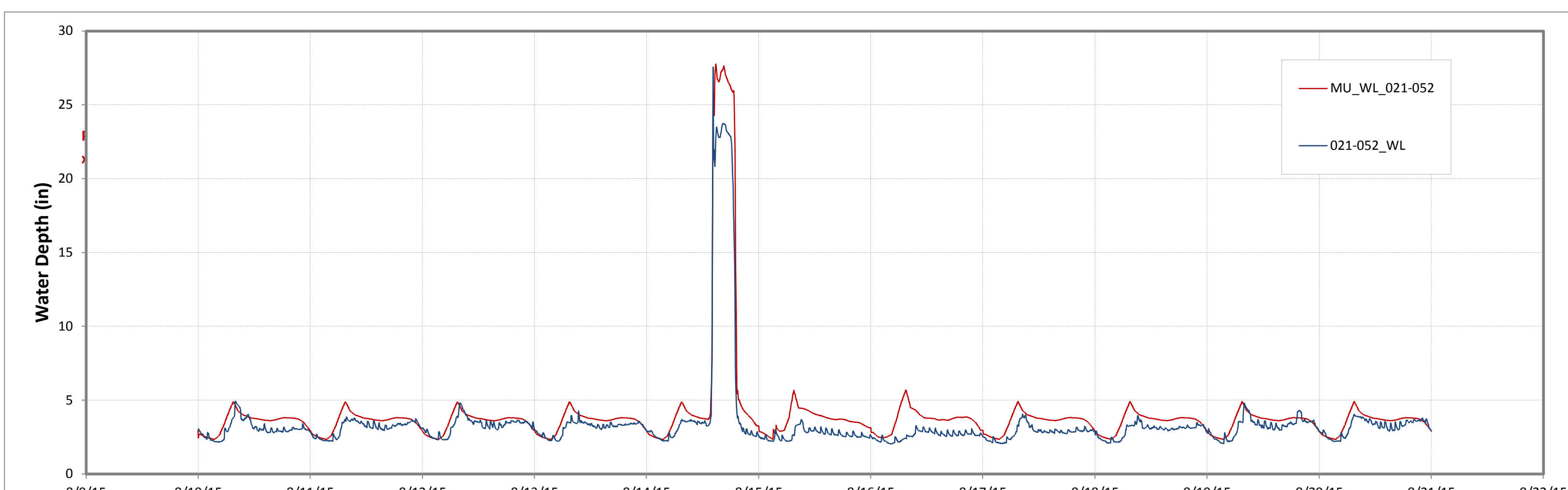


Event 02

07/14/15 to 07/24/15

PEST Optimization	
RDII_AREA	24.9 %
B_A_IFLAT	7.0 %
B_M_IFLAT	0.257
UMAX	3.31 in
LMAX	3.57 in
CQOF	0.23
CK	12.1 hr
CKIF	983.8 hr
CKBF	2114.9 hr

Goodness-of-Fit	
Threshold	0.00
Bias	16.56%
RMSE	0.68
Nash	-1.38
Peak Error	19.85%

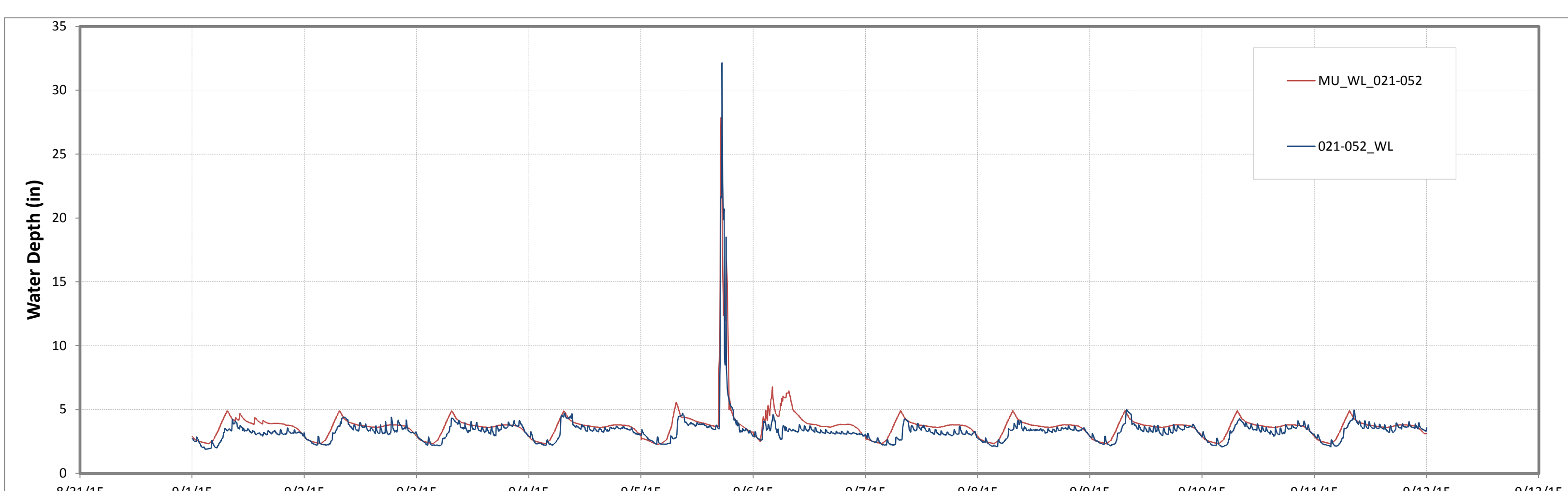


Event 03

08/10/15 to 08/21/15

PEST Optimization	
RDII_AREA	24.9 %
B_A_IFLAT	7.0 %
B_M_IFLAT	0.257
UMAX	3.31 in
LMAX	3.57 in
CQOF	0.23
CK	12.1 hr
CKIF	983.8 hr
CKBF	2114.9 hr

Goodness-of-Fit	
Threshold	0.00
Bias	18.53%
RMSE	1.02
Nash	0.85
Peak Error	0.71%

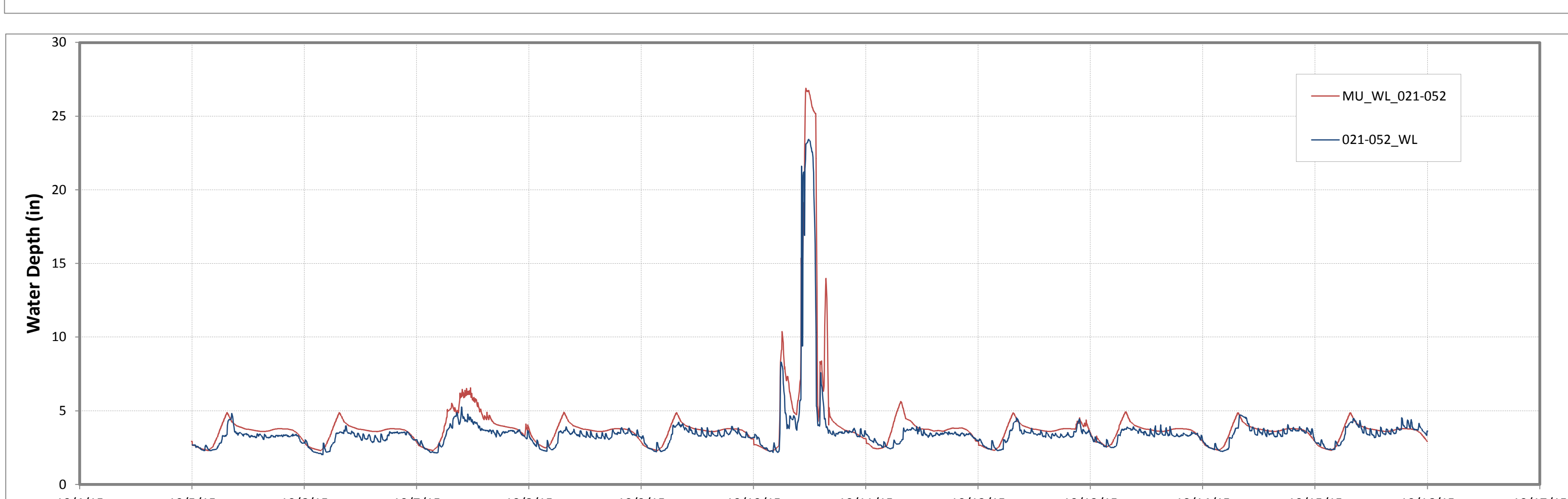


Event 04

09/01/15 to 09/12/15

PEST Optimization	
RDII_AREA	24.9 %
B_A_IFLAT	7.0 %
B_M_IFLAT	0.257
UMAX	3.31 in
LMAX	3.57 in
CQOF	0.23
CK	12.1 hr
CKIF	983.8 hr
CKBF	2114.9 hr

Goodness-of-Fit	
Threshold	0.00
Bias	#N/A
RMSE	#N/A
Nash	#N/A
Peak Error	-13.28%

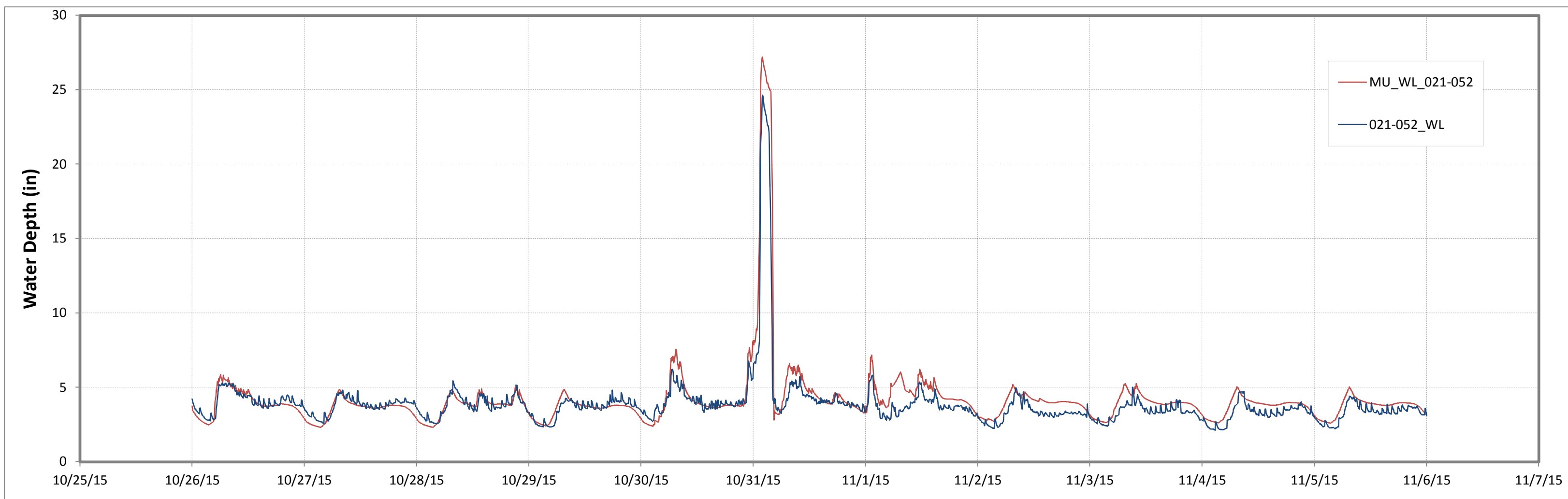


Event 05

10/05/15 to 10/16/15

PEST Optimization	
RDII_AREA	24.9 %
B_A_IFLAT	7.0 %
B_M_IFLAT	0.257
UMAX	3.31 in
LMAX	3.57 in
CQOF	0.23
CK	12.1 hr
CKIF	983.8 hr
CKBF	2114.9 hr

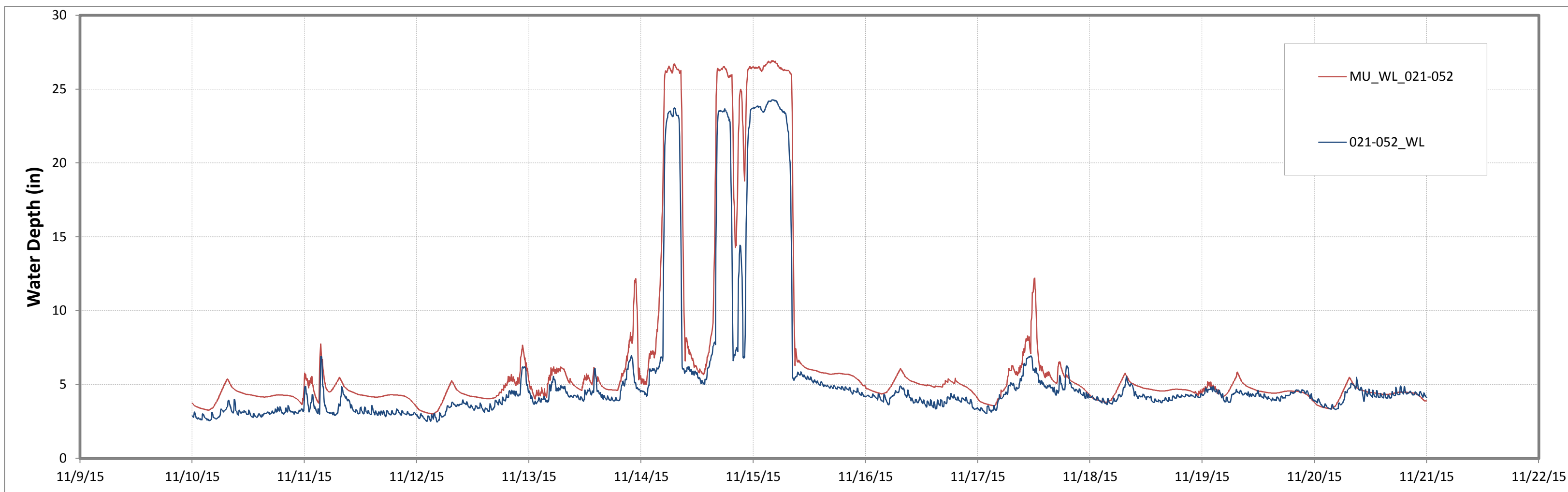
Goodness-of-Fit	
Threshold	0.00
Bias	#N/A
RMSE	#N/A
Nash	#N/A
Peak Error	14.85%



Event 06
10/26/15 to 11/06/15

PEST Optimization	
RDII_AREA	24.9 %
B_A_IFLAT	7.0 %
B_M_IFLAT	0.257
UMAX	3.31 in
LMAX	3.57 in
CQOF	0.23
CK	12.1 hr
CKIF	983.8 hr
CKBF	2114.9 hr

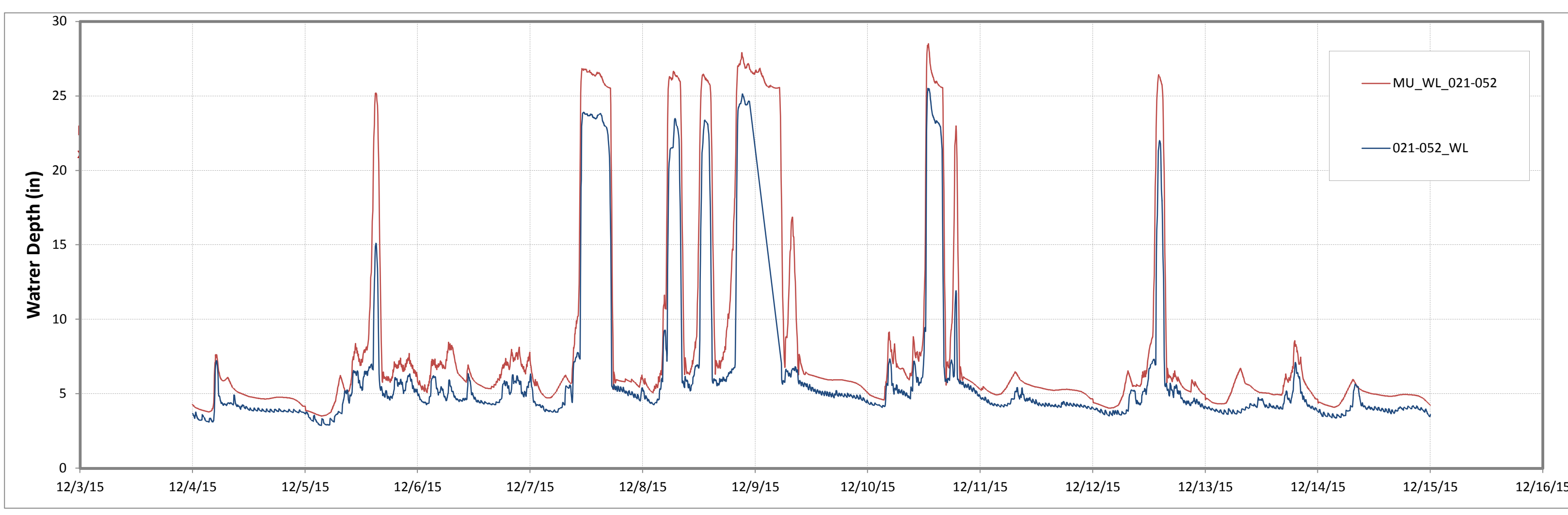
Goodness-of-Fit	
Threshold	0.00
Bias	8.45%
RMSE	0.87
Nash	0.80
Q_{pk} Error	10.32%



Event 07
11/10/15 to 11/21/15

PEST Optimization	
RDII_AREA	24.9 %
B_A_IFLAT	7.0 %
B_M_IFLAT	0.257
UMAX	3.31 in
LMAX	3.57 in
CQOF	0.23
CK	12.1 hr
CKIF	983.8 hr
CKBF	2114.9 hr

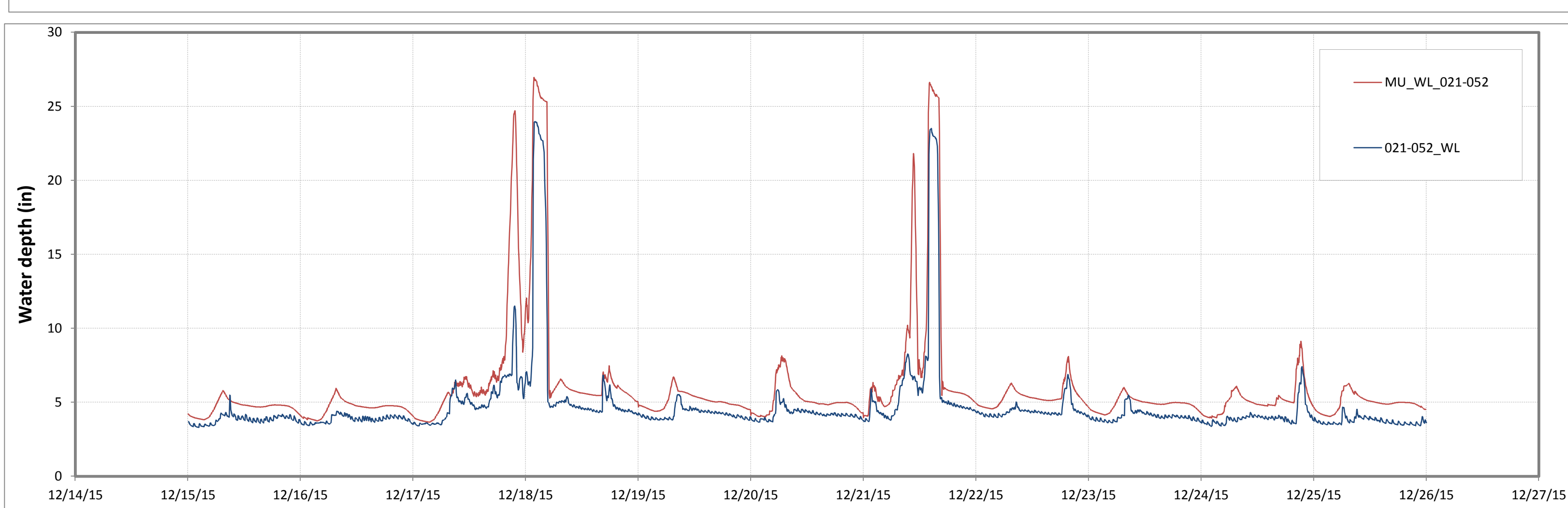
Goodness-of-Fit	
Threshold	0.00
Bias	20.78%
RMSE	2.02
Nash	0.82
Peak Error	10.99%



Event 08
12/04/15 to 12/15/15

PEST Optimization	
RDII_AREA	24.9 %
B_A_IFLAT	7.0 %
B_M_IFLAT	0.257
UMAX	3.31 in
LMAX	3.57 in
CQOF	0.23
CK	12.1 hr
CKIF	983.8 hr
CKBF	2114.9 hr

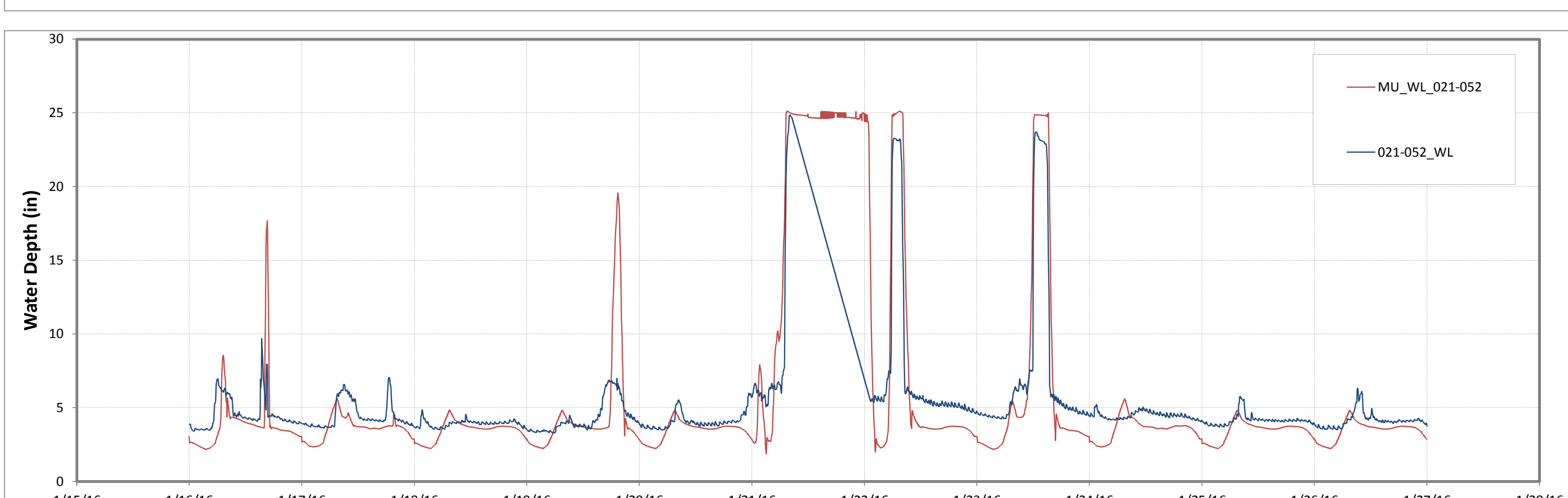
Goodness-of-Fit	
Threshold	0.00
Bias	#N/A
RMSE	#N/A
Nash	#N/A
Peak Error	11.81%



Event 09
12/15/15 to 12/26/15

PEST Optimization	
RDII_AREA	24.9 %
B_A_IFLAT	7.0 %
B_M_IFLAT	0.257
UMAX	3.31 in
LMAX	3.57 in
CQOF	0.23
CK	12.1 hr
CKIF	983.8 hr
CKBF	2114.9 hr

Goodness-of-Fit	
Threshold	0.00
Bias	#N/A
RMSE	#N/A
Nash	#N/A
Peak Error	12.43%



Event 10
01/16/16 to 01/27/16

PEST Optimization	
RDII_AREA	24.9 %
B_A_IFLAT	7.0 %
B_M_IFLAT	0.257
UMAX	3.31 in
LMAX	3.57 in
CQOF	0.23
CK	12.1 hr
CKIF	983.8 hr
CKBF	2114.9 hr

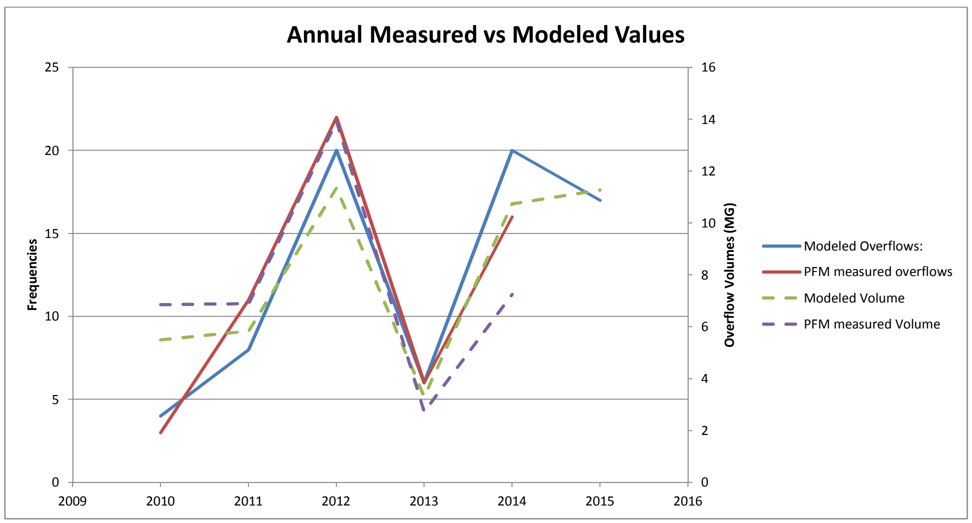
Goodness-of-Fit	
Threshold	0.00
Bias	#N/A
RMSE	#N/A
Nash	#N/A
Peak Error	1.02%

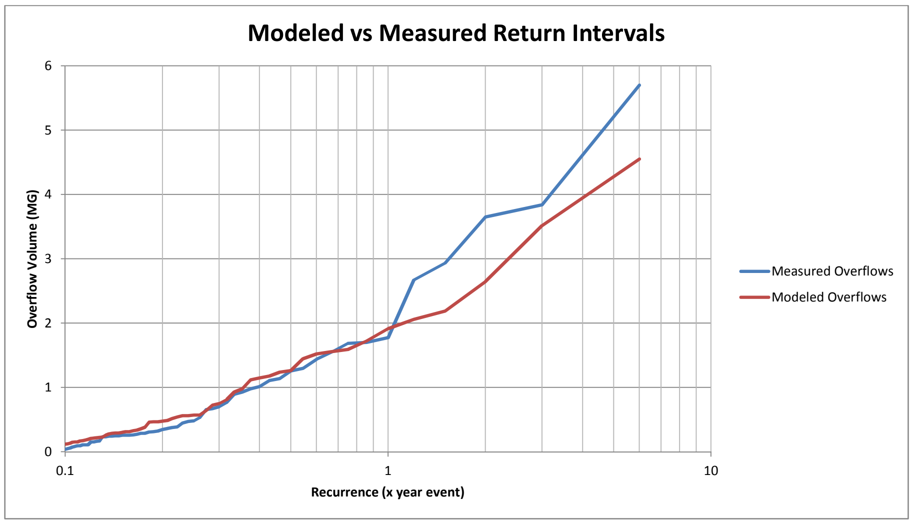
FRE_174, Post-weir retrofit

13.1 Date Range is 10/3/2010-1/1/2016

Frequencies	**2010**	**2011**	**2012**	**2013**	**2014**	**2015**	**AVERAGE**
Modeled Overflows:	4	8	20	6	20	17	12.5
SPU Reported (measured) overflows	4	10	17	7	20	15	12.2
PFM measured overflows	*3*	*11*	*22*	*6*	*16*		*11.6*
Difference	0	-2	3	-1	0	2	0.3
% difference	0%	-20%	18%	-14%	0%	13%	3%
Overflow Volumes (MG)	**2010**	**2011**	**2012**	**2013**	**2014**	**2015**	**AVERAGE**
Modeled Volume	5.49	5.83	11.36	3.30	10.75	11.28	8.00
SPU Reported (measured) Volume	6.49	5.88	10.26	2.78	8.76	13.56	7.95
PFM measured Volume	*6.85*	*6.90*	*13.93*	*2.73*	*7.25*		*7.53*
Model Difference	-1.00	-0.05	1.10	0.53	1.98	-2.28	0.05
% difference	-15%	-1%	11%	19%	23%	-17%	1%

Hydrologic parameters:	174v13			
Hydraulic Version	1			
HLC at 021-055:	0.05	Modeled 1 -year event, 2010-2015:	1.92	MGD
Manning's n at L_021-052	0.0133	SPU Reported (measured) event, 2010-2015:	1.78	MGD
Orifice height	3.75	*PFM measured event, 2010-2015:*	*1.86*	*MGD*
Orifice Invert:	120.33	Difference:	0.14	
Other notes:		% difference:	8%	





Appendix C. Calibration and Model Development Report for Ballard’s MIKE URBAN Model

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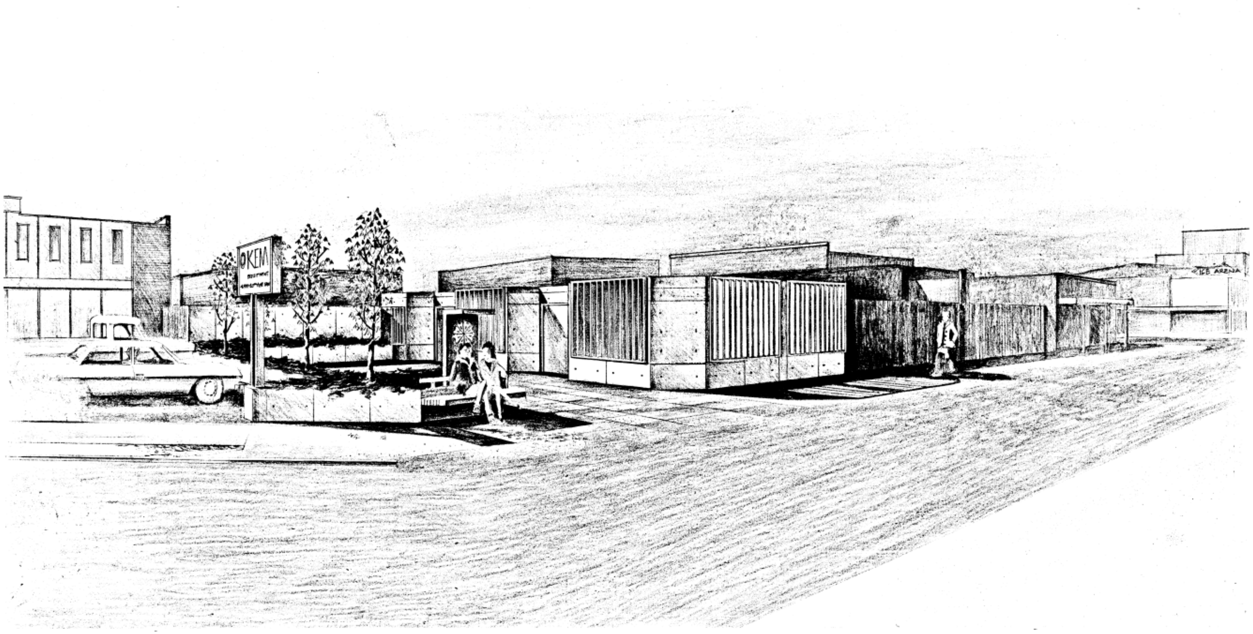
Calibration and Model Development Report for Ballard's MIKE URBAN Model

Patrick Vandenberg

January 22, 2018



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List of Abbreviations

CSO	Combined Sewer Overflow
CSS	Combined Sewer System
DHI	Danish Hydraulics Institute
DWF	Dry Weather Flow
FRC	Fast Response Component
GIS	Geographic Information System
KC	King County
LTCP	Long Term Control Plan
MGD	Million gallons per day
MH	Manhole
NPDES	National Pollutant Discharge Elimination System
RDII	Rainfall-Dependent Infiltration and Inflow
RMSE	Root-Mean-Square Area
RTC	Real Time Control
SCADA	Supervisory Control And Data Acquisition
SCWQP	Ship Canal Water Quality Project
SPU	Seattle Public Utilities
SRC	Slow Response Component
WTD	Wastewater Treatment Division

1 Introduction

This report documents the development and calibration of hydrologic and hydraulic models for the Combined Sewer System (CSS) for the neighborhood of Ballard in the city of Seattle and its downstream connection to King County's Ballard Regulating Station (RS). This area includes two SPU combined sewer overflow (CSO) outfall points, NPDES 150/151 and NPDES 152, which discharge into Salmon Bay. These two outfall points combine for an average of over 50 overflow events per year. SPU previously developed EPA SWMM5 hydrologic and hydraulic models as part of their 2015 Long Term Control Plan (LTCP). Information gained and lessons learned from that modeling effort were used in the current effort to develop and calibrate a model of the basin and overflow structure using the municipal wastewater modeling software MIKE URBAN, developed and distributed by the Danish Hydraulics Institute (DHI). MOUSE is the hydrologic and hydraulic engine within the Mike Urban software. The results of this modeling effort contributed to the development of an Integrated Model with the King County North Interceptor and will contribute to planning, design, construction and operation of the joint Ship Canal Water Quality Project.

1.1 Model Development Approach

The area upstream of the Ballard RS has been further divided into sub-basins for ease of calibration using data from a number of temporary flow meters. ArcMap enabled the use of GIS layers of the various pipes and manholes to delineate each sub-basin based on the area of the network contributing to flow at each meter. These meters were installed in the areas for a similar model development and calibration by Seattle Public Utilities (SPU) and generally gathered data from September 2008 to March 2010. These sub-basins are used as hydrologic modeling units and most were calibrated using quality checked data from a unique portable flow meter. These sub-basins were imported into the municipal wastewater modeling software MIKE URBAN, developed and distributed by the Danish Hydraulics Institute (DHI), where they were assigned hydrologic parameters and connected to the network of pipes, manholes, and control structures in the area. The appropriate flow meter data was then used to calibrate the hydrologic parameters for each sub-basin, a process automated using the calibration program PEST. Section 4 will describe the hydrologic calibration process in greater detail.

1.2 Service Area and Operation

The Ballard CSO Area encompasses 1,088 acres of Seattle. It is bounded by Salmon Bay to the south, NW 85th St. to the north, approximately 15th Ave. NW to the east, and approximately 33rd Ave. NW to the west. Sub-basins 001a, 001b, 002, 003a and 003b include the part of the CSS that is fully combined. The remaining sub-basins are partially separated, meaning that the stormwater is, in some areas, conveyed via a different pipe than is the sanitary sewage.

In the Ballard section of the 2015 LTCP by SPU, the area was divided into two main basins, NPDES 150/151 and NPDES 152, corresponding to the SPU CSO outfall points for each basin and named for the National Pollutant Discharge Elimination System (NPDES) permits associated with the outfalls. For the purposes of this report, additional basin area was added downstream of the confluence of these two basins in order to model the flow all the way to the entrance to the Ballard Regulator. The entire area was further subdivided into 11 sub-basins based on the locations of suitable flow meter data. Section 3.1 discusses the sub-basins in greater detail.

2 Data Sources

Data was collected from a variety of sources to develop the hydrologic and hydraulic models. Hydrology for the basin was characterized from photography, contour data, evaporation, rainfall, and existing delineations of hydrologic basins (from King County's *Runoff-Transport* model). Hydraulics for the collection system were defined based on as-built drawings, GIS sewer coverages, SCADA, and flow meters. Additionally, portions of the SPU modeling report were used as secondary data sources or to verify inferences and approximations.

2.1 Aerial Photography

Detailed 2012 ortho-rectified aerial photographs procured by King County were overlain in GIS to assess basin land use, which was used in part to characterize basin hydrology. Additionally, a map from the SPU modeling report describing the zoning permits of each parcel of the basins was used to verify land use estimates. The basin is zoned mostly as single- and multi-family residences, with some areas zoned for commercial and industrial in the southernmost portion of the basin. This data was used to estimate an expected value for impervious area for each sub-basin.

2.2 Contour Data

GIS overlays of contour data were used to qualitatively estimate the slope of pipe systems within Ballard. Due to the generally uniform slope of the basin, this was generally only used as an order of magnitude verification of the survey data for the pipes and manholes.

2.3 GIS

Point-based shape files of sewer manholes, and line-based files of sewer pipes, were the primary source of the network data used to parametrize the hydraulic model network. Manhole data includes name, diameter of the pipe intersected, and elevations of the rim and of each inlet and outlet pipe. Pipe data include length, diameter, upstream and downstream invert elevations, and material. Polygon-based shapefiles of KC-WTD CSO basins provided the foundation for the basins used in the hydrologic model, and ArcMap was used to create sub-basins based on flow directions as well as to calculate the area of each sub-basin.

2.4 Evaporation

A long-term evaporation record was downloaded from the AgWeatherNet Washington State University Puyallup site. This record was averaged by month into an average evaporation year, which was repeated for the duration of the calibration and long-term period of simulation. Both this data source and methodology are commonly used for continuous hydrologic modeling in the Puget Sound area.

2.5 Rainfall

Processed rainfall records from 1978 to 2015 were provided by SPU at 17 rain gauges located within the City of Seattle. These records consist of a continuous one-minute time series of rainfall depth, adjusted to Daylight Saving Time. Rain gauge RG08 is located just south of the basin and provided the rainfall record for sub-basins 1b, 2, 10, 11, 12a, 12b, and 12c in the hydrologic model.

Sub-basins 1a, 3a, 3b, and 3c were closer to RG07, just north of the basin, and a comparison determined that using this rain gauge provided a better rainfall pattern during calibration than did RG08. The locations of rain gauges relative to Ballard are shown in Figure 2-1 below.

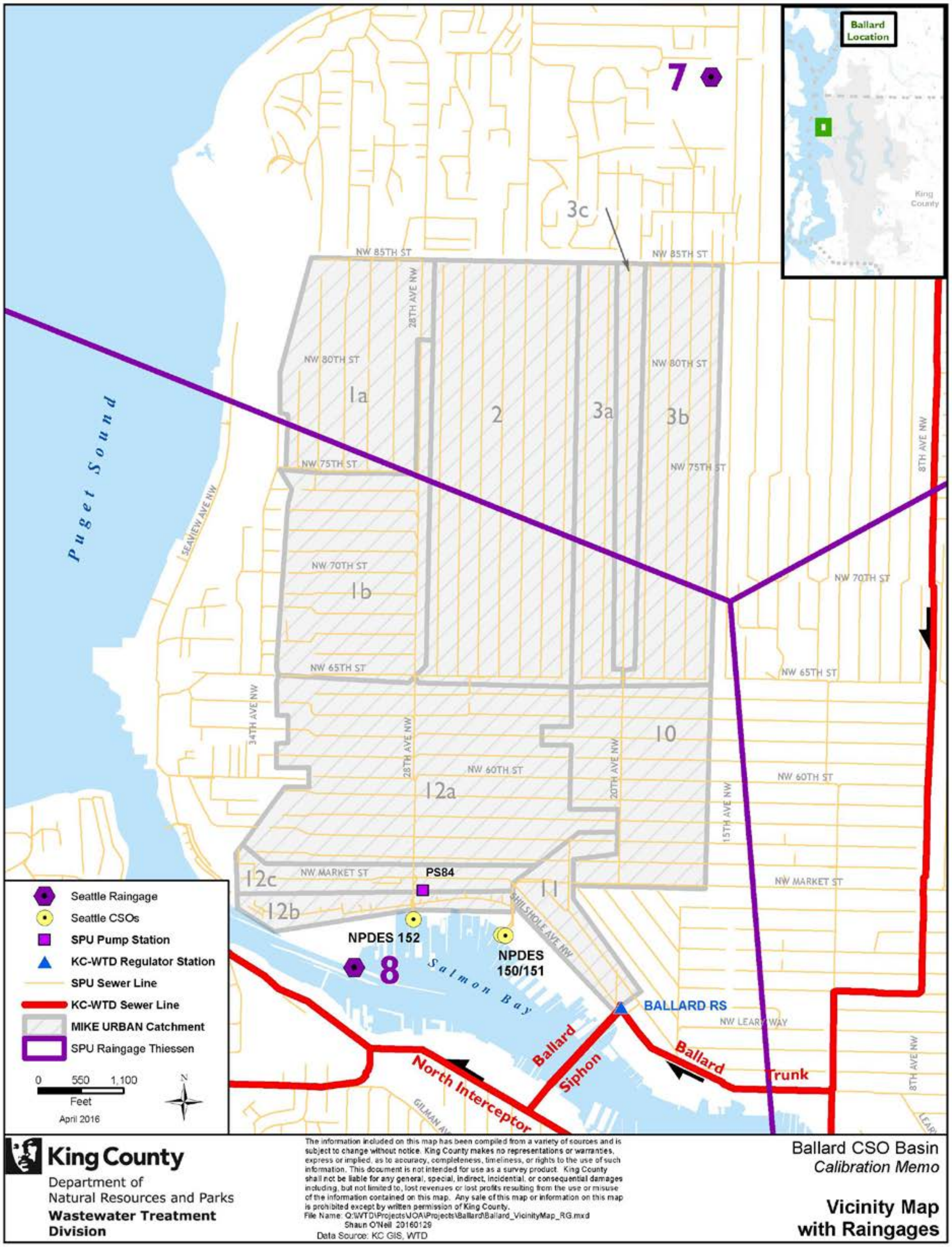


Figure 2-1: Rain gauges and Thiessen polygons in Ballard CSO Area

2.6 As-Built and Design Engineering Drawings

Archived record drawings of sewer pipes and facilities were available from both WTD and SPU. As-Built and design drawings were used to provide modeling detail for facilities and pipes and to address discrepancies and missing data from GIS shape files. These schematics were especially crucial in accurately modeling the physical parameters of the overflow structures.

2.7 SCADA

WTD maintains a historical record of SCADA (Supervisory Control and Data Acquisition) continuously collected from WTD offsite facilities. SCADA data useful for model calibration includes calculated and measured flows, water levels, gate positions, and operational states of pumps and flow regulating structures. SCADA data was used to define a time series for the downstream boundary condition using the trunk water level at the inflow to the Ballard RS.

2.8 Flow Monitoring

WTD and SPU perform conveyance system monitoring with an array of permanent and portable flow meters. These are generally installed in manholes, and have collected upstream depth and velocities for at least one wet season, from which flow rates can be calculated. Data is typically sampled at either 5 or 15 minute intervals. The data are used for the hydrology model calibration and for the verification of the hydraulic model.

For this project, the data used was the same as that used for the SPU modeling report. A more detailed description of the flow meter data is contained in said report. Table 2-A below shows the meters and periods used for the hydrologic calibration, while Figure 2-2 on the following page shows their locations within the basin.

Table 2-A: Flow meter data quality and locations

Flow Meter	Sub-Basin	Data Quality	Start Date	End Date	Address
002-016	1a	Excellent	9/11/2008	9/13/2010	NW 75th Street and 28th Avenue NW
002-032	1b	Excellent	9/12/2008	9/13/2010	NW 65th Street and 28th Avenue NW
002-123	2	Good	9/11/2008	3/15/2010	NW 65th Street and 28th Avenue NW
002-273	3a	Good	9/16/2008	3/2/2010	NW 65th Street and 20th Avenue NW
002-274	3b	Good	9/11/2008	3/2/2010	NW 65th Street and 20th Avenue NW
011-160	12a	Excellent	10/1/2009	3/15/2010	56th Street and 28th Avenue NW in Rotary
011-176	10	Excellent	10/1/2008	2/9/2010	NW Market Street and 20th Avenue NW
011-218	12b	Fair*	9/10/2008	3/31/2009	NW 54th Street and 28th Avenue NW
011-242	11	Excellent	9/12/2008	3/15/2010	NW Vernon Place and Shilshole Avenue NW

*Ultimately, flow meter data from 011-218 was not used for calibration

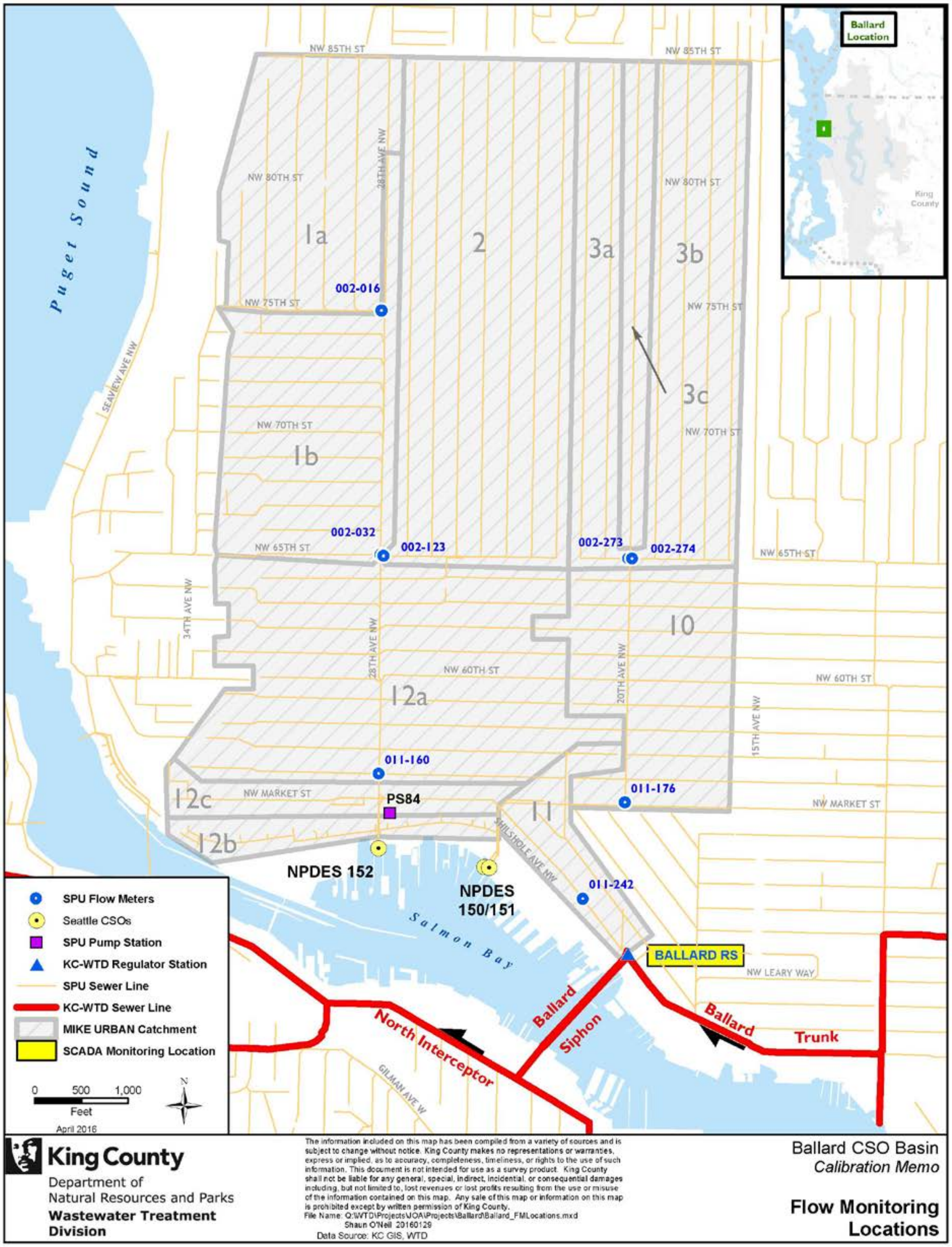


Figure 2-2: Locations of flow meters

3 Model Development

The Danish Hydraulic Institute (DHI) MIKE URBAN model was selected to perform hydrologic and hydraulic modeling of the Ballard CSS. The model consists of a hydrologic component and hydraulic component.

Model development consisted first of compiling the different elements of the model (basins, manholes, pipes, and control structures, which, in the case of this particular basin, included the two overflow weirs), connecting them, and assigning them properties consistent with their physical attributes. Second, rainfall, evapotranspiration, dry weather flows (DWF), and Salmon Bay and Ballard RS trunk water levels were used to define the forcing data and boundary conditions for the model. Third, the head losses at nodes were adjusted to better match observed data and to ensure model stability, as the default parameters in MIKE URBAN tend to compute losses that are higher than monitoring data would suggest are realistic.

Figure 3-1 below highlights the modeled conveyance piping, the control structures, the overflow points, and the connection to the Ballard Regulating Station.

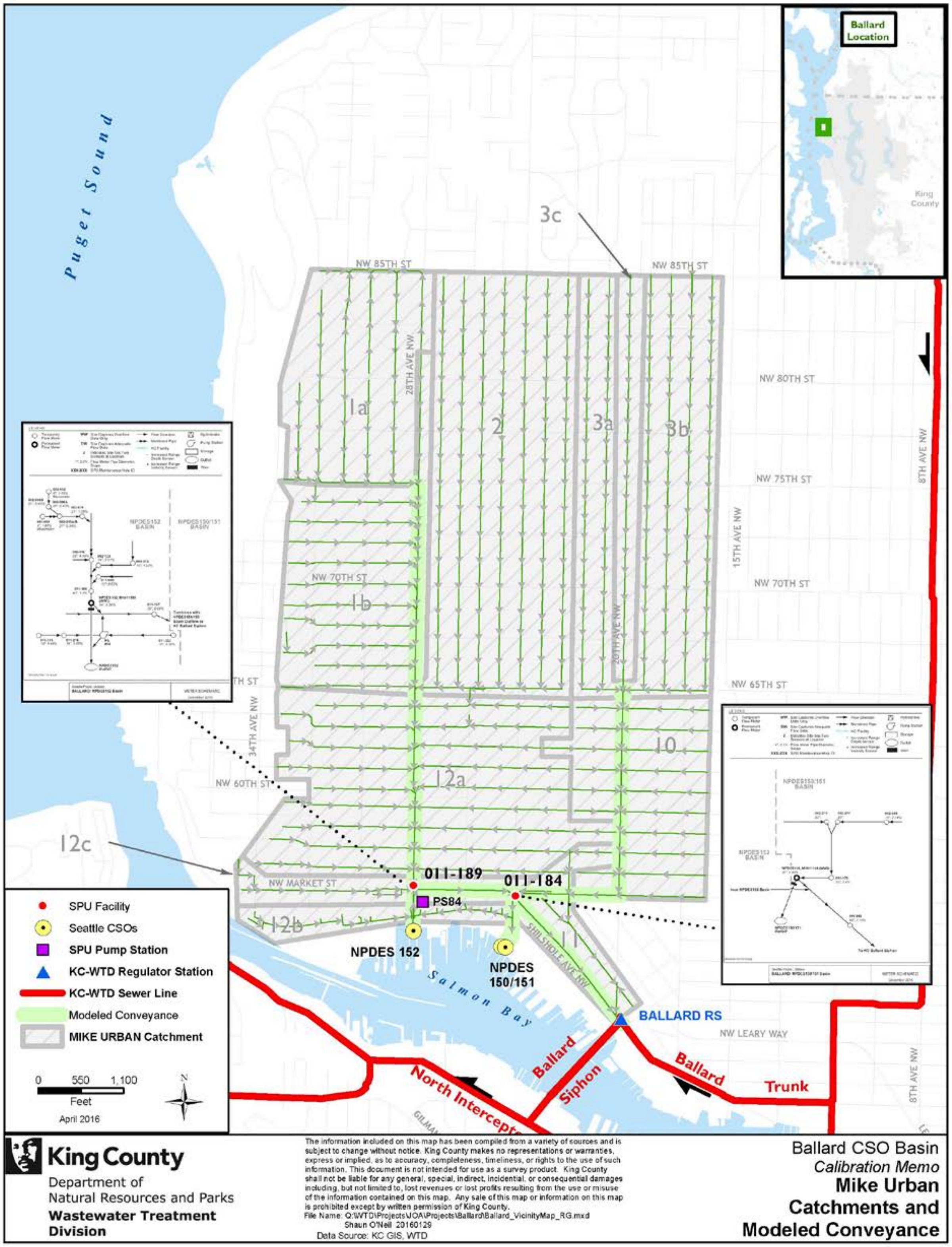


Figure 3-1: Modeled catchments and piping

3.1 Hydrologic Model

The hydrologic component (MOUSE RDII [Rainfall Dependent Infiltration and Inflow] and MOUSE RD Runoff Computation) characterizes the basin response to rainfall in terms of a hydrograph at a tributary location. The hydrograph consists of both surface runoff and RDII. Surface runoff routing is calculated using the Kinematic Wave model (Model B), and accounts for runoff produced from impervious areas of the basin. It's often referred to as the fast response component (FRC). The RDII module accounts for overland flow, interflow, and groundwater processes related to pervious areas of the basin. It's often referred to as the slow response component (SRC). RDII considers evapotranspiration, soil moisture, and groundwater table depth in its calculations.

The process of delineating each sub-basin began by using the previously-defined basins for WTD's *Runoff-Transport* model of the Ballard area, which was based on flow directions throughout the network. Several of these original basins were divided in order to create basins that could be represented with the available flow meter data. The drainage area of each basin, as well as the relevant flow meter, is given in Table 3-A below.

3.1.1 Summary of Sub-Basins

Table 3-A below summarizes the sub-basins calibrated along with their respective areas, associated rain gauge, and flow meter used for calibrating the sub-basin (entries in *italics* were not calibrated but were nonetheless delineated due to some other modeling constraint). Additionally, some of the sub-basins or flow meter data sets were characterized by some unusual circumstances. These are described in greater detail below. Note: the section headings below reflect the catchment names used in the MIKE URBAN model, where SB001a is Sub-Basin 1a, etc.

Table 3-A: List of Model Basins

Sub-Basin	Basin Area (acres)	Rain Gauge	Flow Meter
1a	108.47	RG07	002-016
1b	121.21	RG08	002-032
2	234.39	RG08	002-123
3a	65.187	RG07	002-273
3b	124.05	RG07	002-274
3c	*38.368*	*RG07*	*none*
10	98.598	RG08	011-176
11	*34.228*	*RG08*	*none*
12a	205.81	RG08	011-160
12b	*25.533*	*RG08*	*none*
12c	*31.779*	*RG08*	*none*

3.1.1.1 SB001a, SB001b

The data from the two flow meters 002-016 and 002-032 experienced a pattern of high flow measurements as a result of the potable water flushing station located upstream of 002-016. More details on this phenomenon, as well as the method of removing the flows from the flow timeseries, are included in Section 3.4.3 below.

3.1.1.2 SB002

Data from flow meter 002-123 during the dry weather periods did not follow the expected dry weather flow (DWF) curve. As a result, the diurnal pattern used was less accurate than that for other basins. However, due to the small effect that DWF has on storm flows, this issue was deemed minimally consequential for the purposes of calibration and, as such, a workaround was not investigated.

3.1.1.3 SB003a, SB003b, SB003c

The two flow meters in these basins, 002-273 and 002-274, were located directly upstream of two of the three inlet pipes to the junction at MH 011-322. By analyzing the sewered area in the entirety of sub-basin 003, it was determined that 15% of the sub-basin area enters the junction through the third inlet pipe that is unaccounted for by either flow meter. This area became SB003c and was given hydrologic parameters identical to those of SB003b (or, in the case of flat impervious area percentage and the associated Manning's roughness, the average of the two adjacent sub-basins SB003a and SB003b) due to land use similarities and the overall quality of fit achieved during calibration of SB003b.

Furthermore, data from flow meter 002-274 exhibited abnormally high flow values for the peak flows in calibration events 07 and 09. Additional analysis revealed that the flow became surcharged during these times, causing a deviation from the ideal flow according to Manning's equation. However, because measurements from the downstream flow meter (011-176) did not exhibit a corresponding spike in flow, it was determined that these high readings were not accurate.

3.1.1.4 SB010

A portion of the data from flow meter 011-176 was not adjusted for Daylight Savings Time (DST). As a result, all observed flow data from 2:00 AM on November 1st, 2009 to about 10:00 AM on November 6th, 2009 needed to be translated 1 hour back (in the past) to be consistent with other flow meters and the model date/time. The 1 hour of missing data this generated at the end of this time period was linearly interpolated from the adjacent data.

Additionally, using the high flow data from 002-274 during the surcharged conditions resulted in low flows for sub-basin 10 at the same points in the calibration period (since data from the upstream flow meters 002-273 and 002-274 were used as boundary conditions for sub-basin 10 calibration); however, this did not significantly impact the suitability of the calibrated parameters.

As with any sub-basin where the flow meter was located upstream of the most downstream node in the sub-basin, only the sewered area contributing to the flow at the location of the flow meter was used to calibrate the sub-basin. For SB010, this amounted to 90% of the basin area. For the purposes of this model, the calibrated parameters are considered valid for the entirety of sub-basin 10.

3.1.1.5 SB012a, SB012b, SB012c

SB012a was calibrated with the flow meter data from MH 011-160, and any area not contained upstream of the flow meter was originally assumed to be similar and as a result was given the same hydrologic properties. However, additional research into the network and the role of the pump station PS84 dictated the need to split the sub-basin into northern and southern portions. As a result, SB012b represents the sewered area that is served by PS84 and enters the network downstream of the overflow weir at MH 011-189 via a force main. This area was initially calibrated using the flow meter in MH 011-218, which yielded somewhat suspect results. Due to the relatively small amount of flow originating from this basin, the quality of this calibration was not crucial to overall model performance, and as a result using the same RDII parameters as SB012a was determined to be adequate. The impervious area percentage, impervious Manning's n, and RDII area parameters were kept from the trial calibration, as these parameters yielded a good fit to the average flow indicated by the flowmeter in MH 011-218. PS84 is described in greater detail in Section 3.4.4.

Similarly, SB012c was treated as a separate sub-basin in order to preserve the correct sewered area flowing into the network at node 011-160 and likewise contributing to flow at the overflow weir at MH 011-189; however, it was not calibrated but was instead imbued with the same hydrologic parameters as 12a. The similarity in basin characteristics and the marginally significant scale of flow originating in 12c compared to 12a enabled this approximation.

3.1.1.6 SB011

Due to its relatively small size, the flow contribution from sub-basin 11 was negligible compared to flow from the upstream sub-basins. As a result, flow meter data from MH 011-242 was used to validate the overall performance of the basin as a whole, but was not suitable to calibrate the sub-basin individually. Due to its close proximity, SB011 was given the same hydrologic parameters as SB012a, and no adverse effects were observed when validating basin performance.

3.1.2 Summary of Hydrologic Parameters

The FRC requires length and slope parameters for each basin. These are set constant in the model to 3% and 200 ft, respectively. The length is representative of the average length runoff follows before entering the CSS. A 3% slope is a reasonable slope considering the prevailing topography of the region. The Manning roughness obtained during the hydrologic calibration will adjust as a reflection of any deviation in the slope and length in each basin from the constant values set in the model. The parameters chosen to be calibrated are provided in Table 3-B and the rest of the hydrologic parameters are set to MIKE URBAN default values.

Table 3-B: Parameters calibrated for the hydrologic model

Parameter Name	Model ID	Description	Units
Impervious Area – Flat	B_A_IFLAT	Fraction of basin area that is impervious and connected to the CSS	%

Parameter Name	Model ID	Description	Units
Impervious Manning Number - Flat	B_M_IFLAT	Defines the roughness of the impervious basin area, used in the hydraulic routing of the runoff (Manning's equation)	N/A
RDII Area	RDII_AREA	Fraction of basin that contributes RDII to the CSS	%
Surface Storage	UMAX	Defines the maximal water contents in surface storage	In
Root Zone Storage	LMAX	Defines the maximal water contents in root zone storage	In
Overland Coefficient	CQOF	Determines the extent to which excess rainfall (after surface storage is retained) runs off as overland flow	N/A
Time Constant	CK	Determines how fast the flow responds to rainfall. Also has some effect on the routing of interflow.	Hr
Time Constant Interflow	CKIF	Together with Umax determines the amount of interflow	Hr
Time Constant Baseflow	CKBF	Determines the hydrograph recession during dry periods	Hr

In addition to these parameters *I_U* and *I_L* (the initial abstractions for surface storage and root zone storage) were fixed at the initial value of *UMAX* and 75% of the initial value of *LMAX*, respectively. The variable *GW_CAREA*, the proportion of the groundwater area to the basin area, was also originally used during PEST calibration, but was later fixed at 100% to ensure model stability.

3.2 Hydraulic Model

The hydraulic component (MOUSE HD Pipe Flow Computation) further characterizes the hydrologic response to rainfall in terms of flow and water levels within the CSS. MOUSE HD Pipe Flow uses a Dynamic Wave model to route flow within a network of nodes and links. Nodes represent manholes or outlets; links represent pipes, storage tanks and control devices such as pumps, valves, weirs and regulated gates. The hydraulic network used to represent the Ballard CSS is highlighted in green in Figure 3-1. Section 3.6 describes the control structures in further detail. The hydraulic model were not calibrated per se; instead, existing dimensions, facility operation manuals, engineering drawings, and monitoring data were used to match the model's hydraulic parameters to physical observations, and adjustments were made where necessary to ensure model stability and realistic results.

3.3 Boundary Conditions

The forcing data for the hydrology are rainfall and evapotranspiration. These have been described in Section 2. The boundary conditions for the hydraulic model are upstream inflow to the CSS, dry weather flow (DWF) hydrographs for each sub-basin, and downstream water levels.

3.3.1 Upstream Inflow

Inflow into the hydraulic model network is generally represented as a time series of metered flows or modeled results. The latter is a common alternative to expanding the network model to upstream basins and networks. Since there are no basins that drain into either NPDES 150/151 or NPDES 152, this approach was only used when calibrating isolated sub-basins and was unnecessary in the aggregate model. When calibrating each individual sub-basin, the flow meter(s) used to calibrate the upstream sub-basin(s) were implemented as a discharge timeseries to provide an upstream boundary condition for the pipe network.

3.3.2 Dry Weather Flows

The DWF is the combination of groundwater infiltration and sanitary flow regularly entering the collection system regardless of precipitation. Meter data during dry weather periods were used to create a set of dry weather diurnal curves for all basins. Three diurnal curves were created for each basin to reflect average observed flows during weekdays, Saturdays, and Sundays. The 5-minute flow meter data was averaged into hourly flow rates to get the 3 DWF patterns for each sub-basin. When available, flow meter data from the dry weather period from 7/14/09 to 8/8/09 was used to determine representative DWF averages.

3.3.3 Downstream Water Levels

At each of the four outlets of the hydraulic model (the three overflow outfalls and the connection to the Ballard Regulating Station), a water level was used as a downstream boundary condition. For each overflow outfall, the Salmon Bay water level was approximated using the average water level in the Ship Canal, 114 ft. as measured by the King County datum. While this value does indeed change with tidal activity and with operation of the Hiram M. Chittenden Locks, the variability does not significantly affect hydraulics in the system, and as such a simplified static value was used to reduce computational expense without undue loss of model accuracy.

At the outlet of the model into the Ballard RS, SCADA data was used to develop a timeseries of the water level in the inlet trunk. Additional modeling using UNSTDY was necessary to extend the timeseries and cover the entirety of the calibration period.

3.4 Pumps and Control Structures

This model includes a few control structures, namely the overflow weirs at MH 011-189 and MH 011-184. Additionally, it considers the effects of, but does not directly model, a potable water flushing station in sub-basin 001a and PS84, an SPU owned and operated pump station serving sub-basin 012b.

3.4.1 MH 011-184 Overflow Weir

In sub-basin 11 there is an overflow weir in MH 011-184, referred to as NPDES 150/151 (so named because, downstream of the weir, the overflow pipe splits into two branches and discharges overflows at two separate locations in Salmon Bay). According to SPU as-built drawings and survey data as well as permanent flow meter data, the weir discharges wastewater to the overflow branch when inflow reaches a depth of approximately 22 inches in the manhole. During the calibration period (September 2009 – March 2011), this weir experienced an average of 25 overflow events per year for an average of 2.8 million gallons (MG) discharged per year. The calibration process for this weir is discussed in Section 5.1.

3.4.2 MH 011-189 Overflow Weir

In sub-basin 12 there is an overflow weir in MH 011-189, referred to as NPDES 152. Like NPDES 150/151, overflows from this point are discharged to Salmon Bay. SPU as-built drawings and survey data show that the weir is approximately 12.5 inches high, which is generally reflected in analysis of the overflow data (transient hydraulic conditions cause some inconsistencies in the overflow data, which can occur at inflows above 9 inches yet not at inflows of 15 inches). During the calibration period, this weir experienced an average of 47 overflow events per year for an average of 34 MG discharge per year. The calibration process for this weir is discussed in Section 5.2.

3.4.3 Potable Water Flushing Station

A potable water flushing station in sub-basin 1a caused a somewhat consistent pattern of anomalous flow values in meter data from MH 002-016, MH 002-032, and to a lesser extent MH 011-160 (the effects are less pronounced the further downstream the meter is from the station). The flushing flows generally occurred either in the early morning or early afternoon, lasted for an hour or two, and added between 0.06 and 0.17 MGD to the ambient flow. This flow was instantly recognizable during dry weather periods but was too small to make any significant difference during most rain events. Additionally, the flow was generally nonexistent during long portions of the wet season, even during dry weather periods. Using the difference between the average flow before the flushing began and the average flow during the flushing period, an approximation for the flushing flow rate was determined and removed from the flow data to create a more predictable flow pattern. The resulting MIKE URBAN model does not include the flushing flows.

Figure 3-2: Raw Flow and Calculated Flushing Flow for flow meter 002-016

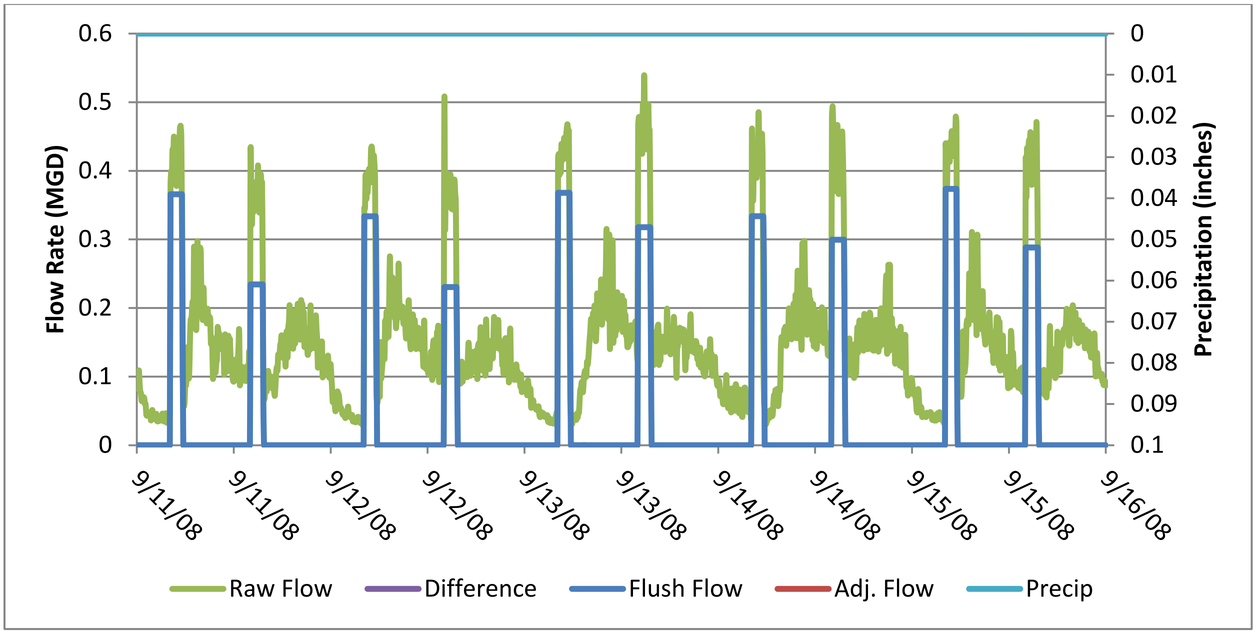
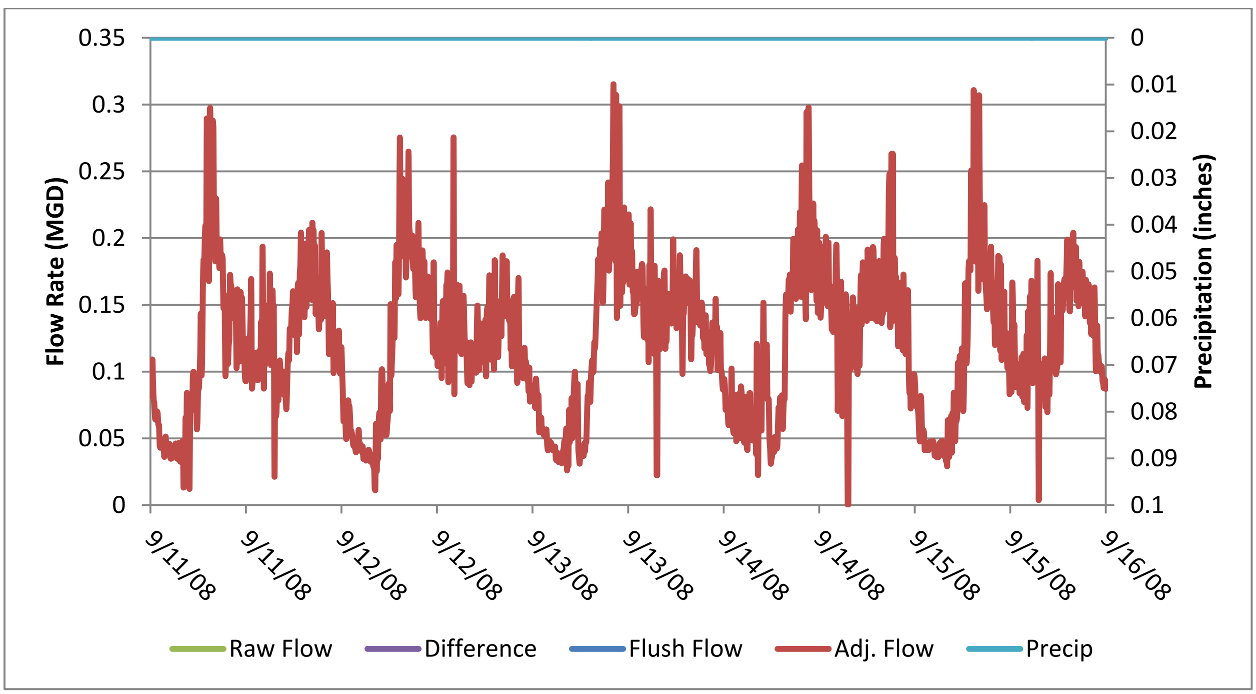


Figure 3-3: Adjusted Flow for flow meter 002-016



3.4.4 PS84

In the center of sub-basin 012b, there is a small pump station referred to as PS84 which serves a single branch of the sewer system and pumps the wastewater into MH 011-193, just downstream of the overflow weir at MH 011-189, via an 8-inch force main. The station includes an overflow structure that would only discharge in the event of a prolonged pump failure. This pump station was not included in the model; however, the area served by the pump station (sub-basin 012b) was separated from sub-basin 012a, and flow from the sub-basin was routed to the network in the appropriate location, downstream of the NPDES 152 overflow weir.

3.5 Losses in Nodes

Head losses in nodes due to manholes and junctions are computed in Mike Urban. The standard calculation usually overestimates the losses, and thus custom losses were set for the nodes in the model. The custom loss estimates follow the Weighted Inlet Energy method (also referred to as Mean Energy Approach), with a coefficient type of total head loss (Total HLC) applied at the outlet of the node. Five standard cases were used in this model. The loss coefficients used are listed in Table 3-C. Some customized losses using the Weighted Inlet Energy Levels (WIE) approach were also applied.

Table 3-C: Head Loss Coefficients

Type of Node	Loss Coefficient
45 Bend	.1
90 Bend	.2
Channelized	.05
Junction	No head loss
Outlet	1

For a few select nodes, the node type was set to “Junction” to manually force the head loss through the node to be zero. This was primarily used in node 011-160, where the MOUSE HD engine was erroneously calculating high head losses without any legitimate cause. This was causing a drop in water level inconsistent with any expectations and prevented the model from accurately modeling overflow events at the manhole immediately downstream, 011-189, where the NPDES 152 overflow structure is located. Additionally, losses in the nodes directly downstream of the overflow structures were increased to help match modeled overflow data to observed.

3.6 Real-Time Control (RTC)

RTC was not used due to the lack of any movable control structures in the basins.

4 Hydrologic Calibration

Model calibration is the process of iteratively adjusting model parameters until the results of the model most closely approximate real world observations. This is done by adjusting the hydrologic parameters of the model for each calibrated basin shown in Table A-2. This iterative process has been automated using Model-Independent Parameter Estimation and Uncertainty Analysis (PEST) software. PEST employs a Gauss-Marquardt-Levenberg method of steepest descent to minimize the differences between model results and observations. Sometimes after the automatic calibration, a manual adjustment of the parameters is carried out to improve the match between model and observations.

Prior to calibration, events are first identified from the monitoring data and assigned appropriate weighting factors. Events with snow are avoided, as it is difficult to model melt rates accurately. Likewise, strong wind events can lead to rainfall measuring errors and should also be avoided. During calibration, the model is run for a sufficient duration to encompass the calibration events. Following calibration, the goodness-of-fit of the results are evaluated using several statistical criterion and metrics.

A spin-up period of at least 2 wet seasons prior to the first calibration event is recommended. For the sake of quality assurance, the hydrologic model was run from the year 2000 until a point shortly after the end of the calibration period. Doing so did not incur significant computational expense.

4.1 Events

Suitable events for calibration were manually selected by reviewing the meter time series. Ten flow events of varying magnitude and duration were identified for each; in general, two events were selected as representative DWF periods, and the remaining eight events were the 8 largest storms in the flow meter date range. Some notable deviations from this approach were:

- Sub-basin 3a: Event 01 (a DWF date range) was truncated to just 3 days due to the later start date of the flow meter data.
- Sub-basin 10: Event 01 (a DWF date range) was selected despite the presence of some precipitation due to the lack of another suitable range during that year.
- Sub-basin 12a: Event 01 (a DWF date range) was selected despite the presence of some precipitation due to the lack of another suitable range during that year. This was the only sub-basin calibrated to a flow meter with data for only a single wet-weather season.

Given the variability in the time span of metered data for each location (see Table A-1), not all basins were calibrated to the same events. The periods used in the calibration are evident in the calibration plots in section 6.

4.2 PEST Weighting

PEST has the ability to weight individual observations during the calibration. PEST will dedicate more computational effort to matching observations with weights above 1, and less effort for weights below 1. Observations with zero weight are effectively ignored.

The selection of an appropriate weighting strategy is dependent largely upon the nature of the calibration and PEST's performance without a weighting scheme. For this particular model, the calibration performed adequately without the use of a variable weighting scheme. All valid values were assigned a weight of 1, and missing or clearly inaccurate data were assigned a weight of 0 to discount their contribution from the calibration error measurement and decision process. The only portion of the time series that were consistently given a weight of 0 was the period from January 4th, 2009 at 13:00 to January 5th, 2009 at 21:00. This precipitation for this timeframe includes snowfall, which resulted in a mismatch in the timing of the response flow and as a result was unfit for calibration purposes.

4.3 Manual Calibration

After the PEST calibration, an analysis of the resulting RDII flow components showed which of those were under- or over-estimated. Adjustment of the parameters based on this information improved the model fit to data for most basins. In general, manual calibration was used to adjust the hydrologic parameter *B_A_Iflat* and was used to attain a more desirable fit for the peak flow values in cases where the model consistently under- or over-estimated the peak flow values.

4.4 Evaluation of Results

The statistical criteria used to evaluate the goodness-of-fit between measured observations and calibrated model predictions for this modeling effort are the standardized Bias, Nash-Sutcliffe model efficiency coefficient, the Root-Mean-Square Error (RMSE), the Relative Peak Flow Difference and Relative Total Volume Difference.

4.4.1 Bias

Bias indicates a general shift of the models, and ranges from positive to negative infinity, where 0.0 indicates a perfect fit. Positive and negative bias indicates model overestimation and underestimation, respectively. Bias is expressed as:

$$B_s = \frac{\sum_{i=1}^{m}\left(Q_i^{mdl} - Q_i^{obs}\right)}{m * \overline{Q^{obs}}}$$

where m is the number of values in the calibration event, Q_i^{mdl} is a model value, Q_i^{obs} is an observed value, and $\overline{Q^{obs}}$ is the average observed value.

4.4.2 Nash-Sutcliffe Coefficient

The Nash coefficient is one less the ratio of the sum of the squared differences between modeled and observed values and the sum of the squared differences between the observed and mean observed values. Nash values can range from 1.0 to negative infinity, where 1.0 indicates a perfect fit. In practice, the Nash coefficient served as the primary goodness-of-fit indicator. Values of 0.9 and greater indicated excellent agreement between the observations and the model. The Nash coefficient is expressed as:

$$Nash = 1 - \frac{\sum_{i=1}^{m} (Q_i^{mdl} - Q_i^{obs})^2}{\sum_{i=1}^{m} (Q_i^{obs} - \overline{Q}_i^{obs})^2}$$

where m is the number of values in the calibration event, Q_i^{mdl} is a model value, Q_i^{obs} is an observed value, and $\overline{Q}_i^{obs}$ is the average observed value in the event.

4.4.3 Root-Mean-Square Error

RMSE is the square root of the average squared difference between the observed and model values. RMSE values can range from 0.0 to positive infinity, where 0.0 indicates a perfect fit. RMSE is expressed as:

$$RMSE = \sqrt{\frac{1}{m} \sum_{i=1}^{m} (Q_i^{mdl} - Q_i^{obs})^2}$$

where m is the number of values in the calibration event, Q_i^{mdl} is a model value, and Q_i^{obs} is an observed value.

4.4.4 Relative Peak Flow Difference and Relative Total Volume Difference

Two additional metrics further quantified the goodness-of-fit. These compared the relative difference of model peak from observed peak, and total model volume from total observed volume. Values of 0.0 for both peak flow and total volume indicate perfect agreement. Positive or negative values indicate model overestimation or underestimation, respectively. The relative peak flow and relative total volume are expressed as:

$$Q_{pk\_diff} = \frac{Q_{\max}^{mdl} - Q_{\max}^{obs}}{Q_{\max}^{obs}} \quad ; \quad V_{diff} = \frac{V_{tot}^{mdl} - V_{tot}^{obs}}{V_{tot}^{obs}}$$

where $Q_{\max}^{mdl}$ is the maximum model value and $Q_{\max}^{obs}$ is the maximum observed value, V_{tot}^{mdl} is the total model volume, and V_{tot}^{obs} is the total observed volume.

4.5 Hydrologic Parameters

The hydrologic parameters calibrated in the model were discussed in Table 3-B. Calibrated values for these parameters are tabulated by basin in Section 6.

4.6 Dates of Snow

There is one time period that could not be used in the calibration because the precipitation fell as snow, causing a time delay between the precipitation measurement and the network flow response to the snowmelt. Because this was a relatively small single occurrence, the period of both precipitation and corresponding flow response (a clear increase in flow corresponding with the melting of the snow) were given a zero weight in the PEST calibration procedure. During calibration, dates between January 4th, 2009 at 13:00 and January 5th, 2009 at 21:00, inclusive, were assigned a weight of 0.

5 Hydraulic calibration

In order to accurately model the frequency and volume of CSO events, the two overflow structures at manholes 011-184 and 011-189 required calibration. Both were characterized by transient hydraulic conditions, which caused difficulties in using MIKE URBAN to model the flow near the structures. Tony Dubin, the lead modeler and coauthor of the SPU LTCP Modeling Report for Ballard, was consulted in order to best understand and model these two structures.

5.1 Outfalls 150/151 Overflow Weir

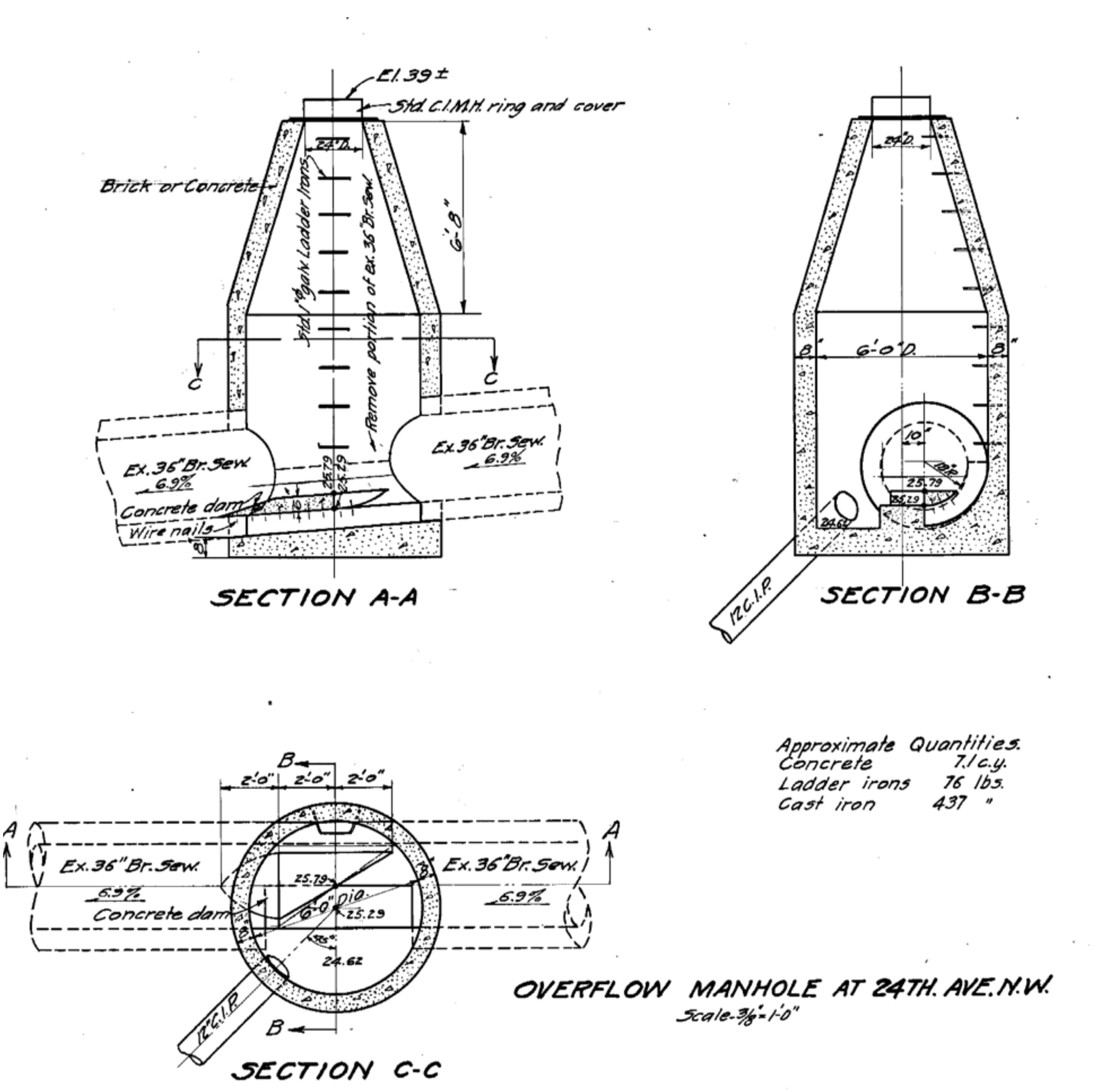


Figure 5-1: Drawing of Outfall 150/151 overflow structure

The overflow structure at MH 011-184 is a broad-crested transverse weir. The inlet and overflow pipes are significantly larger and less steep than the outlet pipe, which has a diameter of 12 inches and dives about 16 feet to connect to the trunk traveling southeast toward the Ballard RS. As a result of this contraction and sudden slope change, the hydraulics of this junction were best modeled by using an orifice for the overflow structure and applying an increased head loss in the outlet pipe and in the connection to the main trunk at the end of the outlet pipe, at node 011-339. In MIKE URBAN, this was accomplished using the parameters summarized in the following table.

Table 5-A: MIKE URBAN Orifice parameters for Outfall 150/151 overflow structure

Weir Type:	Broad Crested Transverse	
Oper. Mode:	No Control	
Weir Crest Elevation:	134.40	ft
Flap:	FALSE	
Discharge coefficient:	1.00	
Height from weir crest to vault ceiling:	9.96	ft
Width of weir:	5.00	ft
L_011-184 Manning's Roughness:	0.0195	
Node 011-339 Head Loss:	3.00	Total HLC

The performance of the modeled overflow structure was compared to observed flow meter data collected by SPU and compiled in the Annual CSO Status Reports. Overall, the model simulated fewer overflow events on average, but managed to match the total volume per year fairly well. One major exception is the storm on 12/2/2007, which was a very large storm that the SPU flow meters at each overflow structure failed to accurately record. As a result, for both overflow weirs, the simulated volume for 2007 is much higher than the recorded volume and was omitted from this summary.

Table 5-B: Outfall 150/151 Overflow Comparison

Outfall 150/151 Overflow Comparison

Year	Metered Overflow Frequency	Modeled Overflow Frequency	Percent Error	Metered Overflow Volume (MG)	Modeled Overflow Volume (MG)	Percent Error
2008	2	5	150%	0.1	0.2	287%
2009	22	15	-32%	3.2	3.0	-5%
2010	29	13	-55%	2.8	2.8	-3%
Total:	**53**	**33**	**-38%**	**6.1**	**6.0**	**-1%**

From a recurrence interval standpoint, the model tends to simulate higher overflow volumes, although it is fairly accurate below 1 year. The upper end of the curve is skewed by the 12/2/2007 storm event, which was not adequately detected by the flow meter used to generate the overflow volume estimates used in the SPU report.

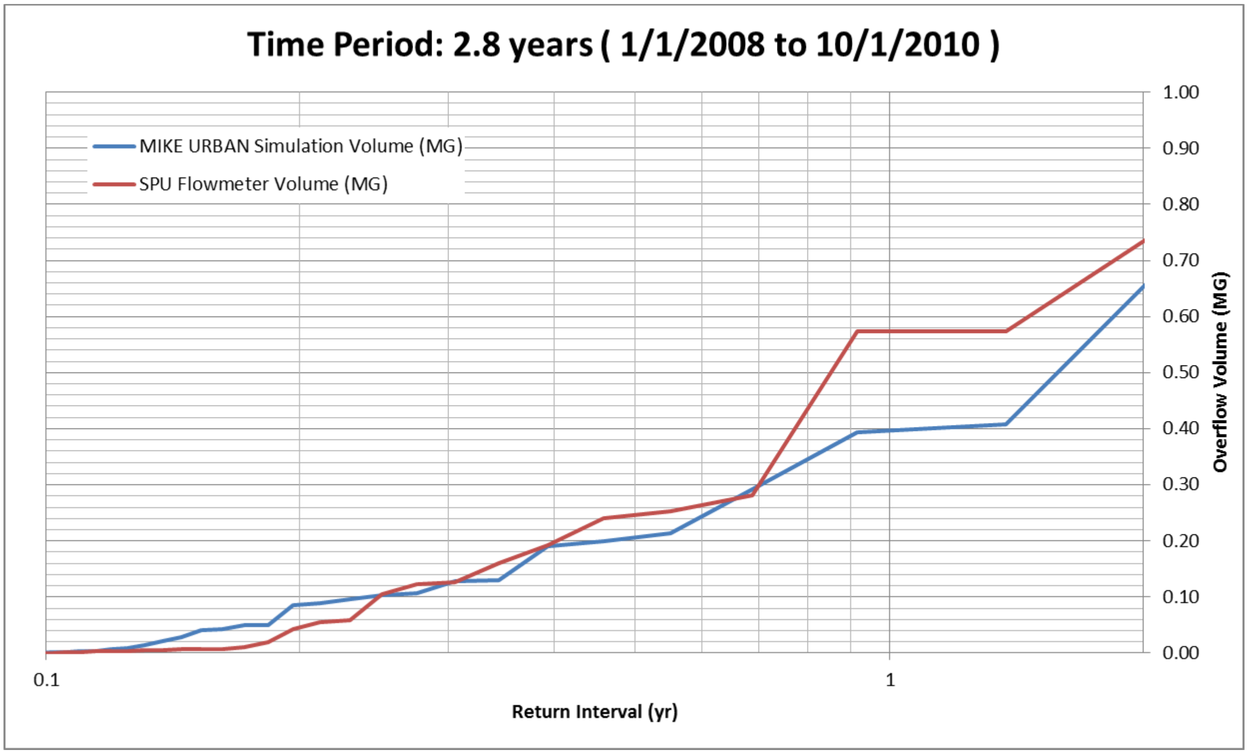


Figure 5-2: Recurrence of Outfall 150/151 overflow volumes

The recurrence interval graphs match poorly above the 6-month return interval point. The 1-year volume of the simulation is about 0.20 million gallons (MG) less than that of the observed data during the 2.8 year simulation period. Because this weir height is not representative of current conditions, a better fit was not pursued at this time. Our designs are based on the greatest instance of the 1-year recurrence interval overflow volume for each 20-year window within the 38-year long-term simulation. Therefore, a 38-year long-term simulation was performed (from 1/1/1978 to 1/1/2016), and the greatest 1-year recurrence interval overflow volume occurred in the 20-year window from 1/1/1996 to 1/1/2016. This value is **0.530 MG**.

The simulated flow depth matches the observed data reasonably well. A comparison of the simulated and observed water levels at the overflow structure is shown in the following graph.

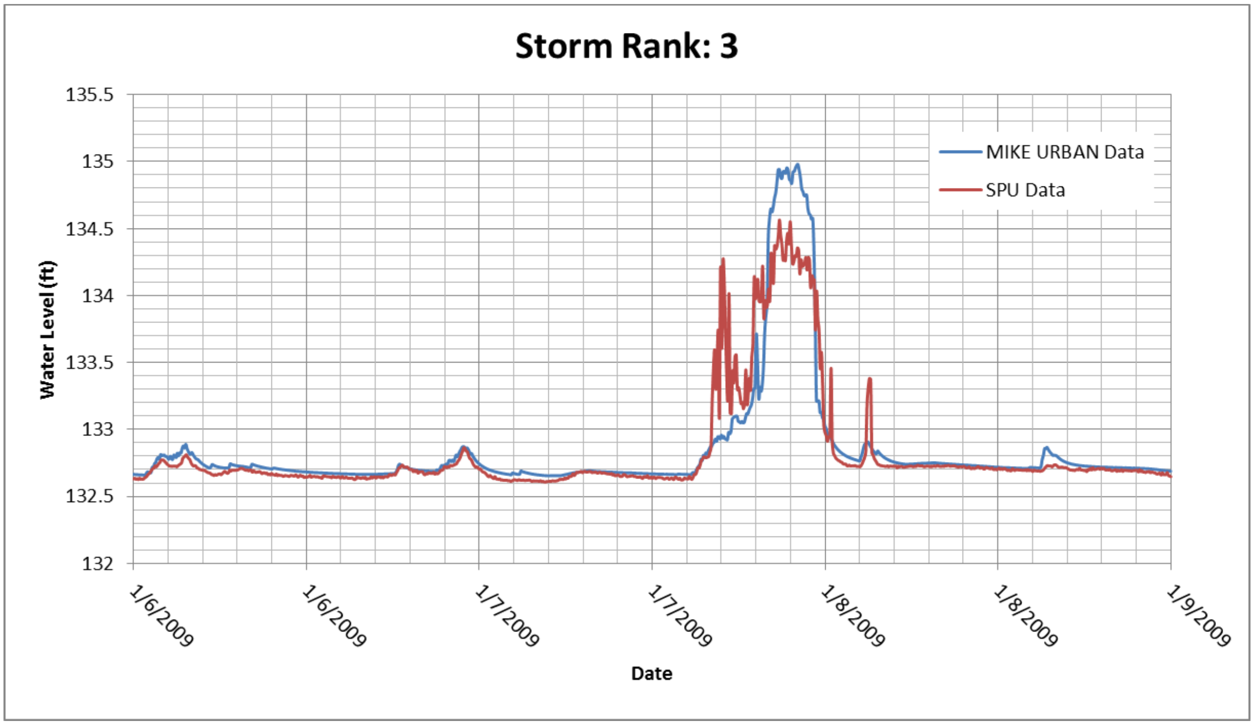


Figure 5-3: Flow elevation for Outfall 150/151 1-year storm

5.1.1 Outfall 150/151 Overflow Weir Retrofit

On October 3rd, 2010, the crest of the overflow structure for Outfall 150/151 was raised by adding a quarter-inch thick angle iron to the top of the structure, from 134.40 ft to 134.73 ft. The effects of this adjustment are shown in the following analysis.

Table 5-C: MIKE URBAN Orifice parameters for Outfall 150/151 overflow structure (post-retrofit)

Weir Type:	Broad Crested Transverse	
Oper. Mode:	No Control	
Weir Crest Elevation:	134.73	ft
Flap:	FALSE	
Discharge coefficient:	1.00	
Height from weir crest to vault ceiling:	9.96	ft
Width of weir:	5.00	ft
L_011-184 Manning's Roughness:	0.0195	
Node 011-339 Head Loss:	3.00	Total HLC

The post-retrofit weir conditions result in similar model performance to that of the pre-retrofit weir conditions. Again, the model misses most of the smaller overflow events and overestimates the flows during the largest ones. As a result, the model simulates only about half of the reported overflows while simulating an approximately similar overall volume. This analysis is shown in the table below.

Table 5-D: Outfall 150/151 (post-retrofit) Overflow Comparison

Outfall 150/151 (Post-Retrofit) Overflow Comparison

Year	Metered Overflow Frequency	Modeled Overflow Frequency	Percent Error	Metered Overflow Volume (MG)	Modeled Overflow Volume (MG)	Percent Error
2010	6	5	-17%	2.0	2.3	14%
2011	25	11	-56%	2.2	2.4	12%
2012	33	15	-55%	4.8	4.6	-3%
2013	12	9	-25%	1.5	2.0	29%
2014	34	22	-35%	3.4	2.9	-16%
2015	35	11	-69%	2.5	2.9	16%
Total:	**145**	**73**	**-50%**	**16.4**	**17.1**	**4%**

However, from a recurrence interval standpoint, the post-retrofit weir conditions perform very favorably. The simulated 1-year overflow volume is 0.644 MG, while the observed is 0.574 MG. Additionally, due to the larger date range for which there is observed data, more confidence can be placed in this analysis than in the analysis of the pre-retrofit conditions. The recurrence intervals are shown on the graph below. The maximum overflow volume in the long term simulation occurred during the same 20-year window as for the pre-retrofit conditions. This value is **0.514 MG**, which is slightly lower than for the pre-retrofit conditions, as expected.

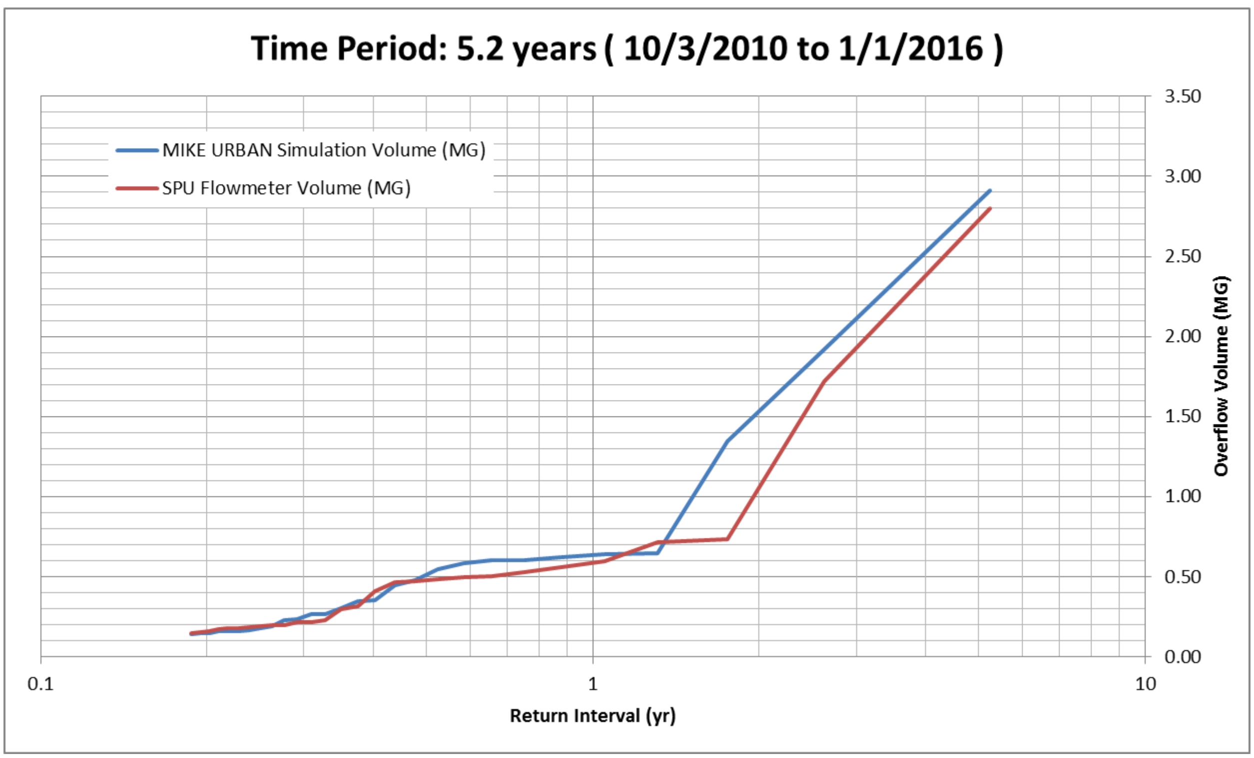


Figure 5-4: Recurrence of NPDES 150/151 (post-retrofit) overflow volumes

The simulated flow depth matches the observed data fairly well. The observed data only records flow in the overflow line, meaning that the flow level was set to the weir crest level when overflows weren't occurring. The graph below shows the simulated and measured flow level for the 1-year event.

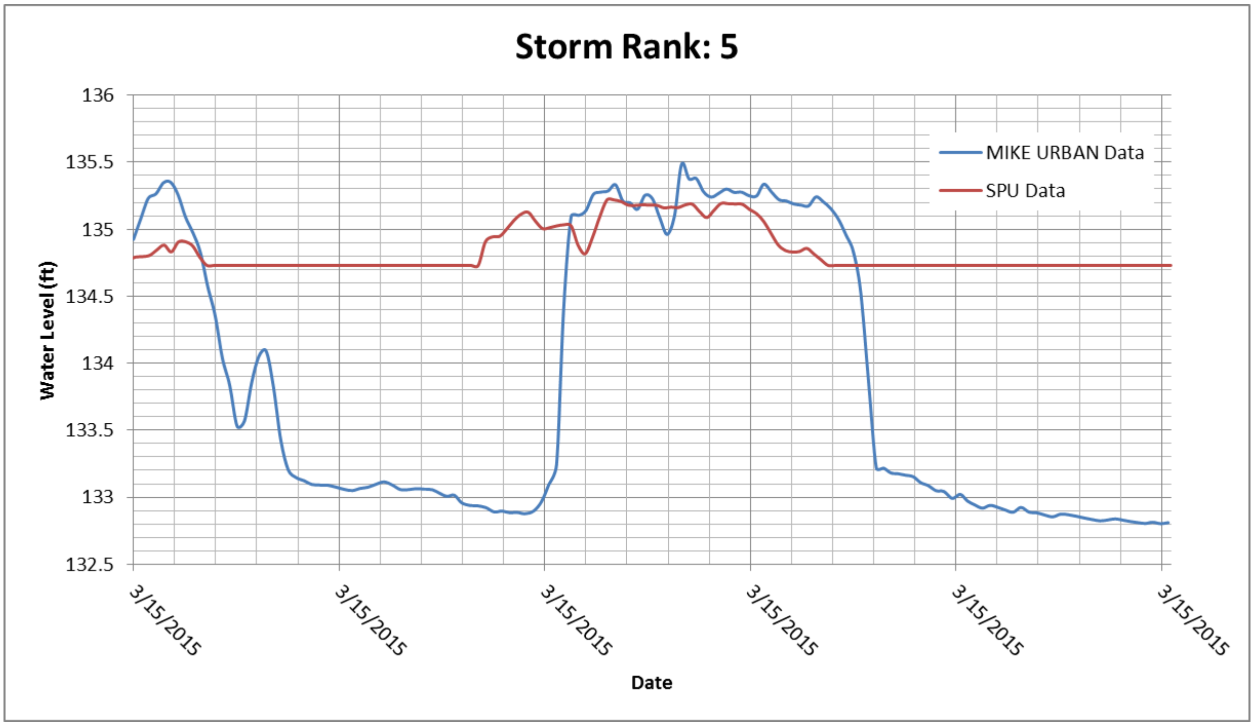


Figure 5-5: Flow elevation for Outfall 150/151 (post-retrofit) 1-year storm

5.2 Outfall 152 Overflow Weir

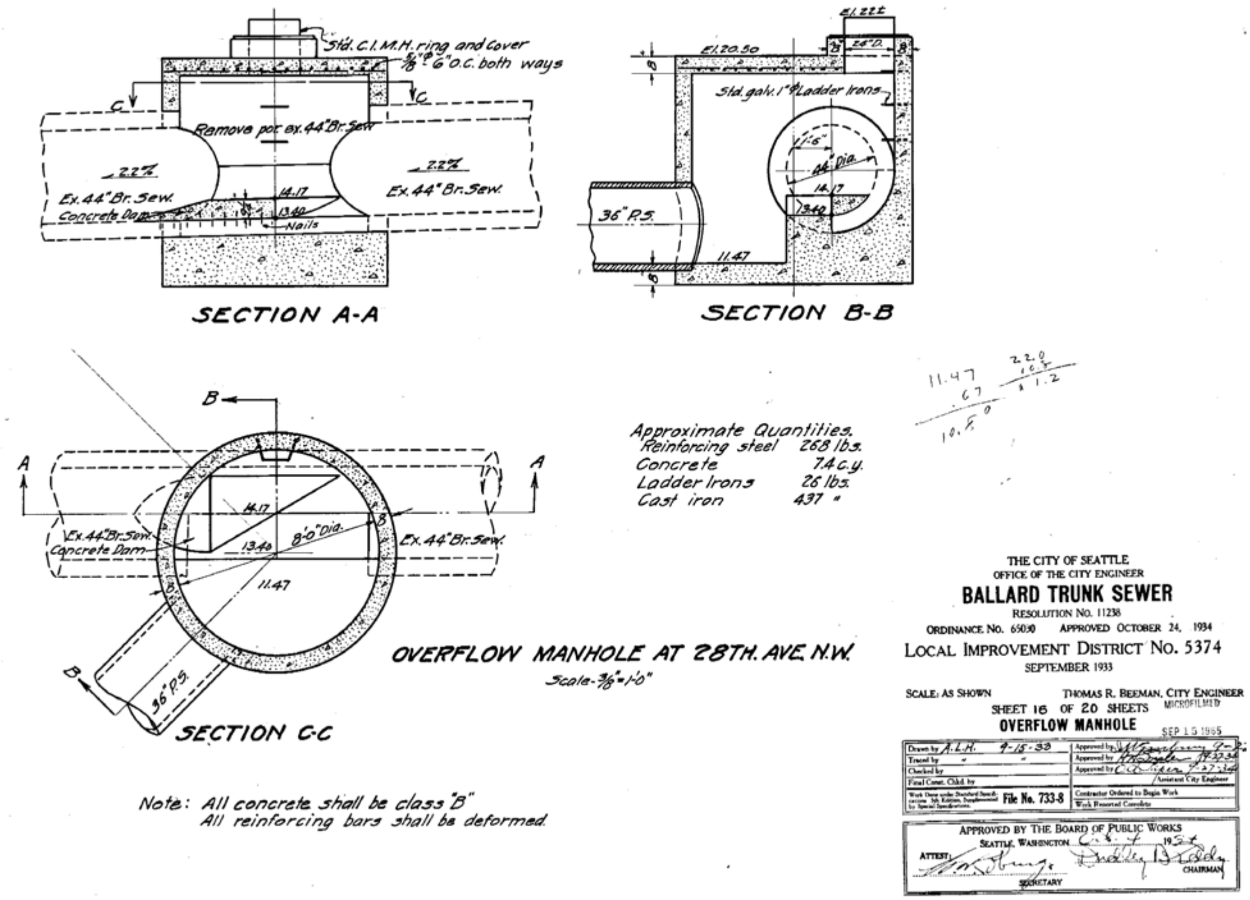


Figure 5-6: Drawing of Outfall 152 overflow structure

After attempting to model the overflow structure at MH 011-189 in a similar approach to the one described above, it became apparent that transient hydraulic conditions caused difficulties for the model when simulating overflows at this location. A sudden change in slope (over 2% in the inlet pipe to under 0.1% just downstream of the outlet) and a 45-degree change in flow direction result in a hydraulic jump occurring near the overflow structure under high flow conditions. This phenomenon was confirmed using videos of low-flow and high-flow conditions. An initial approach to modeling this overflow structure was to use a Q/H relationship based on observed flow meter data. This relationship, which closely resembled the rating curve for a weir, was described in the SPU LTCP Modeling Report as accurately modeling overflows in their SWMM5 model. However, enforcing such a curve caused the MOUSE HD engine to experience errors not easily rectified, and it was determined that the loss of consistent model stability was not sufficiently offset by the increase in accuracy. Consequent attempts to use MIKE URBAN's orifice were greatly facilitated by eliminating the effects of head loss in the node immediately upstream of the overflow weir, and combined with adjusting various head loss coefficients in the outlet pipe, this setup was able to model overflows that more closely match the measured overflow data. The orifice parameters used in the model are summarized in Table 5-C below.

Table 5-E: MIKE URBAN Orifice parameters for Outfall 152 overflow structure

Weir Type:	**Broad Crested Transverse**	
Oper. Mode:	No Control	
Weir Crest Elevation:	120.02	ft
Flap:	FALSE	
Discharge coefficient:	1	
Height from weir crest to vault ceiling:	10	ft
Width of weir:	5	ft
Node 011-188 Head Loss:	3	Total HLC
L_011-189 Manning's:	0.045	
L_011-188 Manning's:	0.02	

A comparison of the MIKE URBAN modeled flow versus the flow meter measured flow for Outfall 152 is summarized in Table 5-D. Note that the 2010-2011 subtotal was used to guide the calibration efforts instead of the 4.5-year total available for comparison. This, as was explained by Tony Dubin, was because the SPU flowmeter consistently underreported the measured overflow discharge before an adjustment in November 2009 fixed this issue. Overall, the model simulated more overflow events with approximately equivalent overall volume in the two years (2010 and 2011) that were used to guide the hydraulic calibration. The frequency and volume of simulated overflows in 2007 through 2009 is much higher than the measured data, which is consistent with Mr. Dubin's explanation of the flow meter shortcomings, and is not included in the summary below.

Table 5-F: Outfall 152 Overflow Comparison

Outfall 152 Overflow Comparison

Year	Metered Overflow Frequency	Modeled Overflow Frequency	Percent Error	Metered Overflow Volume (MG)	Modeled Overflow Volume (MG)	Percent Error
2010	52	67	29%	37.2	45.7	23%
2011	48	40	-17%	40.3	25.3	-37%
Total:	**100**	**107**	**7%**	**77.6**	**71.0**	**-8%**

From a recurrence interval standpoint, determining a 1-year overflow volume from 2 years of observed data is prone to enormous amounts of uncertainty. It overestimates the volume of some of the smaller storms and slightly underestimates the volume of some of the larger ones; however, it grossly underestimates the 1- and 2-year overflow events. For this reason, the recurrence interval graph below had little influence on the hydraulic calibration efforts.

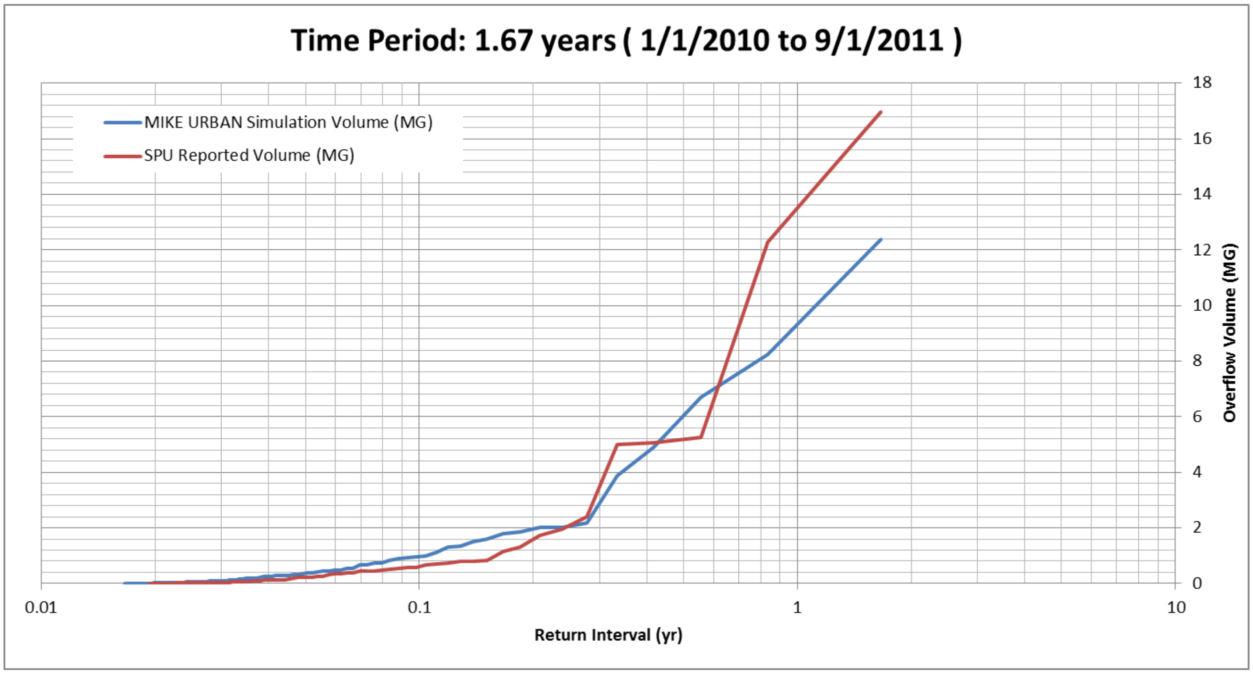


Figure 5-7: Recurrence of Outfall 152 overflow volumes

The graph above shows a large discrepancy, over 4 MG, between the simulated 1-year overflow volume (8.31 MG) and the observed 1-year overflow volume (12.35 MG); however, the short duration of applicable data renders both these estimates very error-prone. When the standard procedure to develop a design volume from the long-term simulation was performed, the greatest 1-year recurrence interval overflow volume occurred in the 20-year window from 1/1/1996 to 1/1/2016. This value is **4.51 MG**, which is much lower than either estimate based on solely 2010-2011.

The simulated flow level matches the observed data fairly well. Uncertainty surrounding the exact elevation of the observed flow depth measurements may have influenced the data, but the two series follow a similar pattern. The 1-year event is shown below.

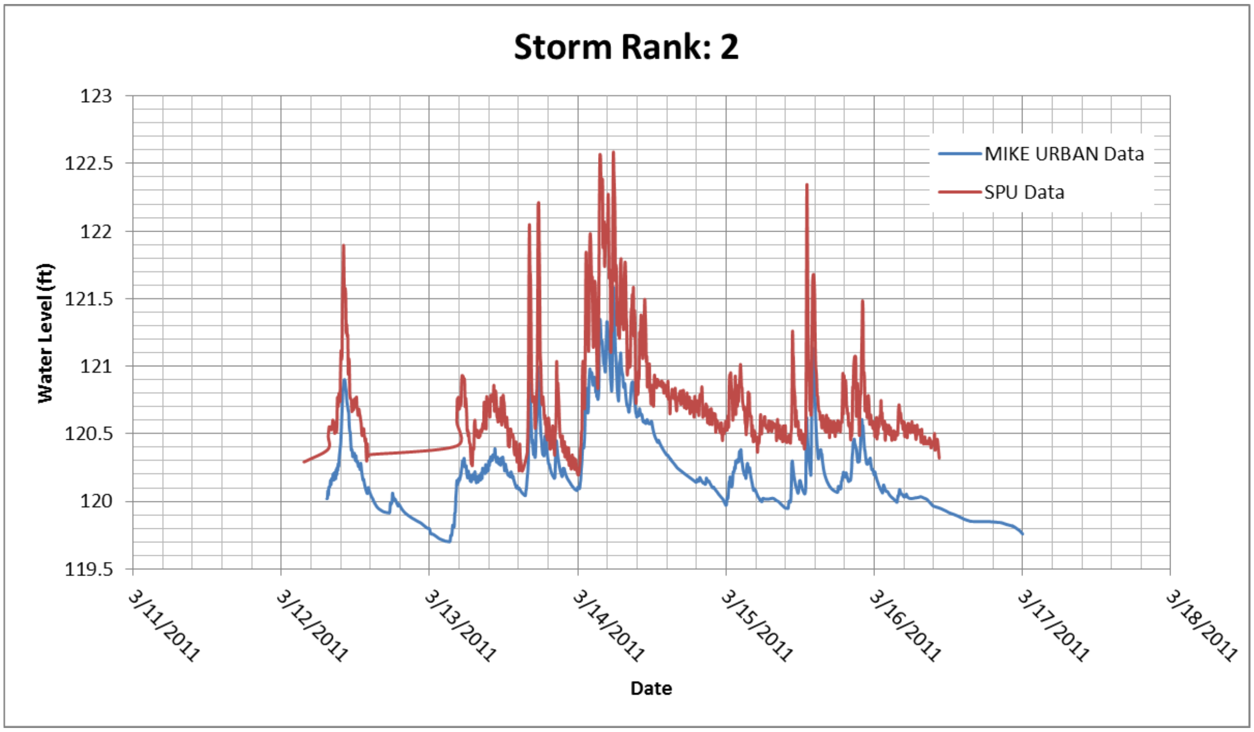


Figure 5-8: Flow elevation for Outfall 152 1-year storm

5.2.1 Outfall 152 Overflow Weir Retrofit

In July/August of 2011, the overflow weir for NPDES 152 was raised approximately 3 inches in an attempt to mitigate the effects of the hydraulic jump that develops near the structure. The following table summarizes the model parameters for the orifice used to model this retrofitted weir.

Table 5-G: MIKE URBAN Orifice parameters for Outfall 152 overflow structure (after weir retrofit)

Weir Type:	**Broad Crested Transverse**	
Oper. Mode:	No Control	
Weir Crest Elevation:	120.24	ft
Flap:	FALSE	
Discharge coefficient:	1	
Height from weir crest to vault ceiling:	10	ft
Width of weir:	5	ft
Node 011-188 Head Loss:	5	Total HLC
L_011-189 Manning's Roughness:	0.05	
L_011-188 Manning's Roughness:	0.04	

This retrofit, along with efforts to improve the accuracy of flow meter data for the overflow weir, provided the physical basis for a much better fitting hydraulic model. Simulated overflow frequency was consistently slightly higher, and simulated overflow volume was consistently much higher than the observed data. A comparison of the MIKE URBAN modeled flow versus the flow meter measured flow for the post-retrofit version of the Outfall 152 is summarized in Table 5-F. Note that the flow meter data from the ADS website was riddled with gaps after August 2015, and as such the comparison did not include figures from that portion of the year.

Table 5-H: Outfall 152 Overflow Comparison (post-retrofit)

Outfall 152 (Post-Retrofit) Overflow Comparison

Year	Metered Overflow Frequency	Modeled Overflow Frequency	Percent Error	Metered Overflow Volume (MG)	Modeled Overflow Volume (MG)	Percent Error
2011	14	14	0%	10.4	9.1	-13%
2012	58	60	3%	51.4	53.3	4%
2013	41	49	20%	12.8	20.2	57%
2014	48	57	19%	32.6	51.9	59%
2015	16	14	-13%	10.6	12.1	14%
Total:	**177**	**194**	**10%**	**117.9**	**146.5**	**24%**

From a recurrence interval standpoint, the simulation volume is lower for the smaller storms and fairly accurate for the larger storms (1-year and larger).

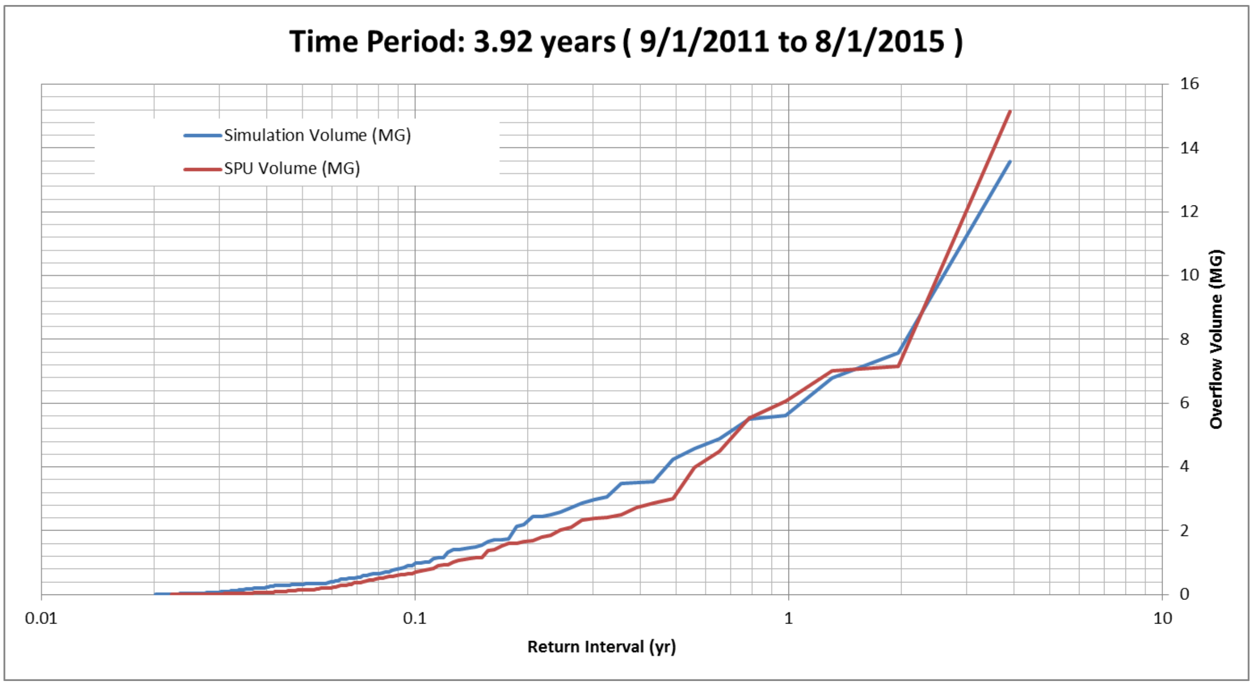


Figure 5-9: Recurrence of Outfall 152 (post-retrofit) overflow volumes

The observed 1-year overflow volume for this time period (6.12 MG) is slightly higher than the simulated 1-year overflow volume (5.68 MG). The largest 1-year recurrence interval overflow volume during the long-term simulation occurred in the 20-year window from 1/1/1978 to 1/1/1998. This value is **4.91 MG**. This value is slightly higher but similar to that of the pre-retrofit weir model.

The simulated flow level matches the observed data fairly well. Again, uncertainty surrounding the elevation of the flow monitoring equipment may be the source of some bias. The 1-year event is shown in the graph below.

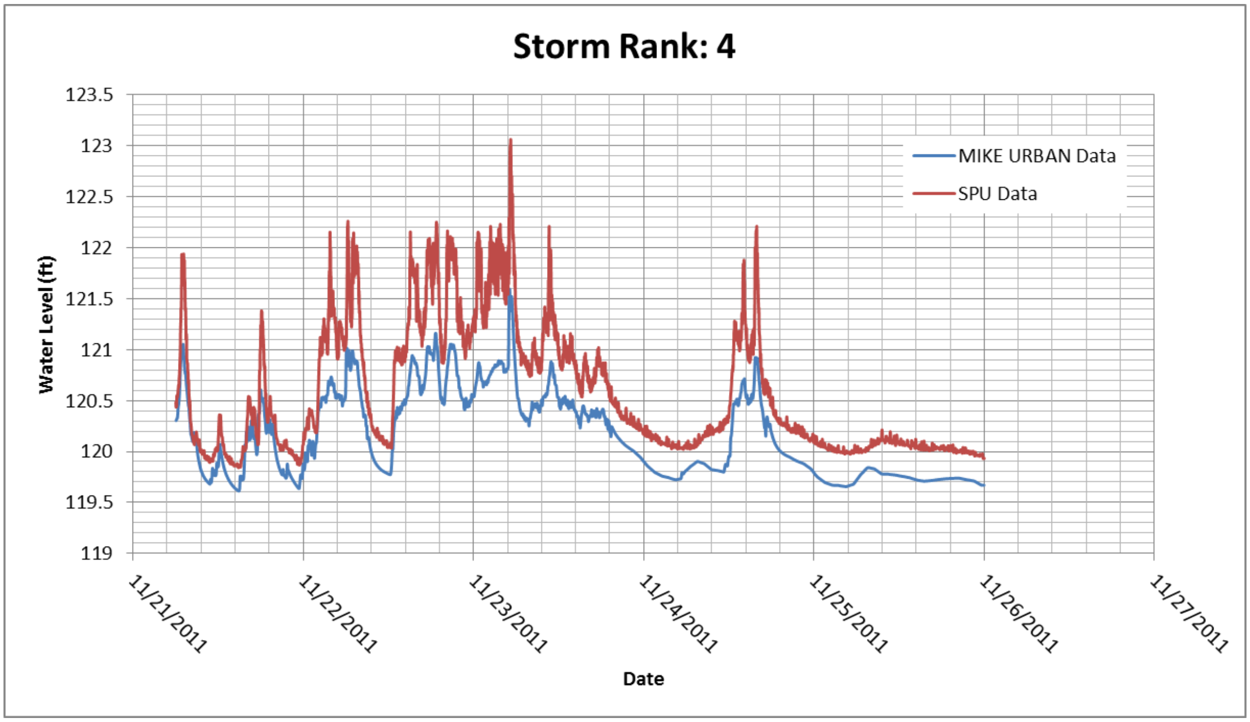


Figure 5-10: Flow elevation for Outfall 152 (post-retrofit) 1-year storm

6 Results

6.1 Hydrologic Calibration

The calibrated parameters for each sub-basin are given in Table 5-A. For each calibrated basin, the evaluation statistics are presented in Tables 5-B through 5-H. Graphs showing the calibrated model output, observed flow, and precipitation for each sub-basin in included in Appendix A.

6.1.1 Calibrated Parameters and Goodness-of-Fit Metrics for each Sub-Basin

Parameters	SB001a	SB001b	SB002	SB003a	SB003b	SB010	SB012a	SB012b
RDII_AREA	49.1 %	78.0 %	16.0 %	7.9 %	27.5 %	114.8 %	45.6 %	26.1 %
B_A_IFlat	30.0 %	14.9 %	25.0 %	23.0 %	32.0 %	18.0 %	7.6 %	3.5 %
B_M_IFlat	0.079	0.064	0.061	0.069	0.072	0.040	0.079	0.035
UMAX	3.40 in	3.93 in	3.94 in	4.66 in	3.92 in	0.20 in	1.67 in	1.67 in
LMAX	7.99 in	6.00 in	4.14 in	5.91 in	5.18 in	13.84 in	6.14 in	6.14 in
CQOF	0.63	0.33	0.76	1.00	0.48	0.22	0.57	0.57
CK	6.1 hr	3.3 hr	6.2 hr	2.7 hr	3.1 hr	17.2 hr	7.9 hr	7.9 hr
CKIF	812.3 hr	458.3 hr	898.1 hr	1188.1 hr	341.9 hr	146.7 hr	193.9 hr	193.9 hr
CKBF	1810.1 hr	853.2 hr	1000.0 hr	100.0 hr	846.2 hr	1700.7 hr	998.2 hr	998.2 hr
I_U	0.39 in	0.39 in	0.39 in	0.39 in	0.39 in	0.39 in	0.39 in	0.39 in
I_L	4.43 in	4.43 in	5.91 in	5.91 in	5.91 in	4.43 in	4.43 in	4.43 in
GW_CAREA	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000

SB001a

Event	Bias	RMSE	Nash	Qpk Error	Vtot Error
01	0.05	0.02	0.76	-18.8%	2.3%
02	0.01	0.18	0.88	-44.1%	-4.1%
03	-0.18	0.33	0.78	-8.5%	-17.2%
04	-0.02	0.19	0.88	-4.9%	-7.3%
05	0.14	0.03	0.74	-12.7%	18.3%
06	0.18	0.13	0.90	31.6%	14.2%
07	0.36	0.40	0.77	47.2%	34.4%
08	0.20	0.22	0.93	19.5%	14.3%
09	0.15	0.16	0.96	-11.9%	8.9%
10	0.03	0.16	0.96	5.5%	2.3%
Average:	**0.09**	**0.18**	**0.86**	**0.3%**	**6.6%**

SB001b

Event	Bias	RMSE	Nash	Qpk Error	Vtot Error
01	0.06	0.03	0.67	-5.0%	7.4%
02	-0.06	0.31	0.89	53.8%	1.3%
03	0.00	0.27	0.95	-14.3%	-4.4%
04	0.26	0.17	0.95	14.6%	15.8%
05	0.21	0.04	0.38	15.4%	7.4%
06	0.19	0.14	0.96	-9.2%	2.0%
07	0.06	0.26	0.95	2.9%	10.0%
08	0.03	0.23	0.97	-23.2%	3.1%
09	0.03	0.20	0.98	2.1%	2.4%
10	-0.04	0.28	0.96	-5.3%	-3.2%
Average:	**0.07**	**0.19**	**0.87**	**3.2%**	**4.2%**

SB002

Event	Bias	RMSE	Nash	Qpk Error	Vtot Error
01	-0.15	0.08	0.39	-34.7%	-24.6%
02	0.29	0.53	0.61	14.7%	41.7%
03	0.22	0.66	0.64	6.6%	11.9%
04	0.30	0.35	0.77	17.8%	38.8%
05	-0.01	0.05	0.64	-28.6%	-5.6%
06	0.02	0.22	0.89	8.7%	12.1%
07	0.03	0.53	0.82	-41.6%	3.0%
08	0.10	0.27	0.95	-5.2%	12.2%
09	0.09	0.32	0.94	-17.7%	10.8%
10	0.07	0.51	0.81	13.3%	9.6%
Average:	**0.10**	**0.35**	**0.75**	**-6.7%**	**11.0%**

SB003a

Event	Bias	RMSE	Nash	Qpk Error	Vtot Error
01	0.19	0.01	0.53	-22.8%	18.1%
02	0.13	0.13	0.75	51.4%	27.0%
03	0.06	0.17	0.71	2.8%	0.5%
04	0.10	0.09	0.81	33.4%	22.7%
05	0.13	0.01	0.68	-7.7%	12.3%
06	0.21	0.06	0.88	18.2%	14.3%
07	0.11	0.11	0.88	-18.7%	0.4%
08	0.04	0.07	0.96	0.5%	4.6%
09	0.02	0.07	0.96	-13.6%	0.4%
10	0.06	0.12	0.88	-2.8%	-1.1%
Average:	**0.10**	**0.08**	**0.80**	**4.1%**	**9.9%**

SB003b

Event	Bias	RMSE	Nash	Qpk Error	Vtot Error
01	0.04	0.03	0.79	-15.1%	-1.7%
02	0.10	0.17	0.90	-14.1%	12.2%
03	0.06	0.37	0.73	8.6%	1.2%
04	0.10	0.22	0.76	62.2%	24.7%
05	-0.09	0.03	0.73	-20.0%	-12.4%
06	0.11	0.16	0.87	31.1%	14.7%
07	0.31	0.78	0.45	-4.0%	38.4%
08	0.25	0.28	0.87	25.8%	23.1%
09	0.10	0.30	0.91	-44.3%	1.0%
10	-0.01	0.16	0.96	-6.9%	7.9%
Average:	**0.10**	**0.25**	**0.80**	**2.3%**	**10.9%**

SB010

Event	Bias	RMSE	Nash	Qpk Error	Vtot Error
01	-0.26	0.20	0.86	-10.1%	-17.3%
02	-0.11	0.59	0.76	1.9%	-1.9%
03	-0.02	0.26	0.97	-6.8%	-6.6%
04	0.05	0.14	0.98	-0.3%	7.1%
05	0.08	0.06	0.88	1.3%	8.7%
06	0.07	0.12	0.98	-2.3%	2.0%
07	0.09	0.53	0.89	72.0%	8.5%
08	0.01	0.19	0.98	-7.8%	0.9%
09	-0.06	0.35	0.96	58.8%	-4.8%
10	0.00	0.24	0.98	6.1%	1.8%
Average:	**-0.01**	**0.27**	**0.92**	**11.3%**	**-0.2%**

SB012a

Event	Bias	RMSE	Nash	Qpk Error	Vtot Error
01	0.01	0.10	0.92	22.2%	6.6%
02	0.00	0.20	0.99	-0.8%	-2.5%
03	0.04	0.46	0.97	3.1%	4.0%
04	-0.01	0.32	0.99	11.0%	0.3%
05	0.00	0.30	0.99	-0.4%	-1.2%
06	0.01	0.25	0.98	-1.2%	-2.1%
07	-0.02	0.35	0.99	7.1%	1.7%
08	0.04	0.25	0.97	16.0%	4.9%
09	0.01	0.16	0.97	11.9%	-1.7%
10	-0.03	0.19	0.98	2.5%	-6.0%
Average:	**0.01**	**0.26**	**0.98**	**7.1%**	**0.4%**

SB012b

Event	Bias	RMSE	Nash	Qpk Error	Vtot Error
01	-0.04	0.01	0.67	-8.9%	-15.3%
02	0.04	0.01	0.69	-7.3%	5.0%
03	-0.06	0.02	0.60	68.3%	-13.2%
04	-0.04	0.01	0.73	4.0%	-13.1%
05	-0.02	0.01	0.60	-4.2%	3.8%
06	0.10	0.02	-0.13	37.6%	28.2%
07	0.07	0.01	0.69	41.7%	4.3%
08	0.02	0.02	0.47	-35.9%	-19.1%
09	0.05	0.02	0.38	4.5%	-4.4%
10	-0.13	0.02	0.46	58.1%	-20.9%
Average:	**0.00**	**0.01**	**0.51**	**15.8%**	**-4.5%**

011-187

Event	Bias	RMSE	Nash	Qpk Error	Vtot Error
01	-0.36	0.48	-0.72	-42.4%	-37.6%
02	-0.16	0.69	0.80	26.2%	-13.0%
03	-0.15	0.84	0.60	15.3%	-8.5%
04	-0.28	0.65	0.53	22.6%	-22.8%
05	-0.37	0.52	-1.52	-35.8%	-37.1%
06	0.00	0.39	0.85	16.2%	2.8%
07	-0.04	0.67	0.87	13.4%	5.0%
08	-0.20	1.09	0.80	-10.7%	-18.4%
09	-0.18	0.85	0.84	-2.1%	-19.6%
10	-0.24	1.37	0.58	-8.0%	-25.9%
Average:	-0.20	0.75	0.36	-0.5%	-17.5%

SB011

Event	Bias	RMSE	Nash	Qpk Error	Vtot Error
01	-0.03	0.14	0.88	-15.2%	-5.2%
02	0.10	0.87	0.90	24.1%	12.1%
03	0.04	1.17	0.81	10.1%	0.6%
04	0.04	0.48	0.92	12.6%	7.2%
05	-0.03	0.14	0.88	-9.3%	-2.5%
06	0.11	0.56	0.91	12.4%	12.6%
07	0.18	0.97	0.90	21.4%	22.5%
08	0.06	0.61	0.97	12.9%	6.0%
09	0.03	0.50	0.98	3.9%	-0.7%
10	-0.04	0.77	0.95	12.9%	-4.5%
Average:	**0.05**	**0.62**	**0.91**	**8.6%**	**4.8%**

6.1.2 Remarks on Hydrologic Calibration

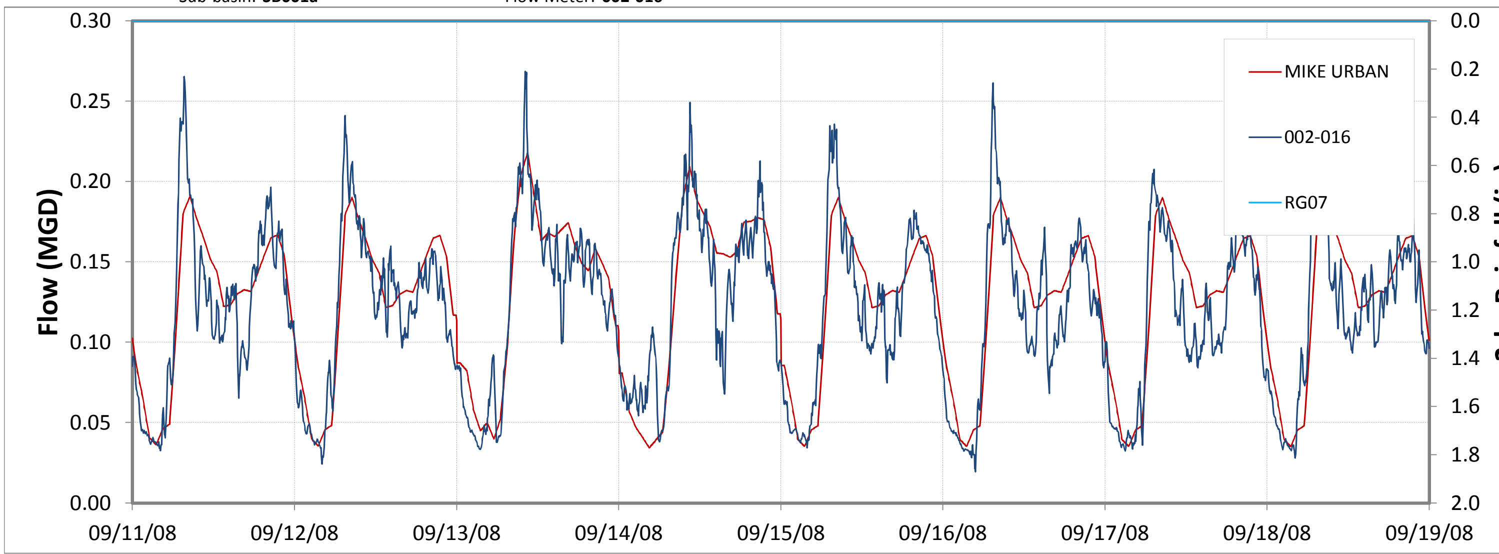
Overall, PEST was able to find parameter sets for each sub-basin that resulted in good-fitting simulated hydrographs. With a few noted exceptions, the Nash-Sutcliffe coefficient was generally very high for the wet-weather storms. The peak flow was more often underestimated than overestimated for many of the sub-basins. This may be due to the basins being underrepresented by the rain gauges, and at any rate modeled output should be used as a low-end estimate for peak flow. Storm volume and time to peak were much more accurate. It is the opinion of this modeler that the model is suitable for estimating flows within the system and approximating overflow volumes over long periods. The model is not useful in simulating the size of individual overflow events, and any attempt to use the model to accurately simulate the number of events should be viewed as an upper bound

Potential additions to the model would be in inclusion of newer flow meter data for calibration, addition of green stormwater infrastructure (GSI) projects, or any sweeping changes in land use.

6.1.3 Calibration Graphs

The following graphs show the simulated flow, flow meter, and rain gauge time series used to calibrate each sub-basin, as well as the calibrated hydrologic parameters and various goodness-of-fit metrics. Note that the calibration graph for SB012b is called “PS84” as it represents the flow passing through that pump station. Additionally, the simulated vs. measured flow at MH 011-187 is included in the following graphs even though the flow meter was not used for calibration. This comparison includes some inherent error during dry weather periods, as the sub-basins SB012b and SB012c do not contribute to the dry weather flow in the model. However, due to the accuracy seen in the verification plot in SB011, this was kept in the model.

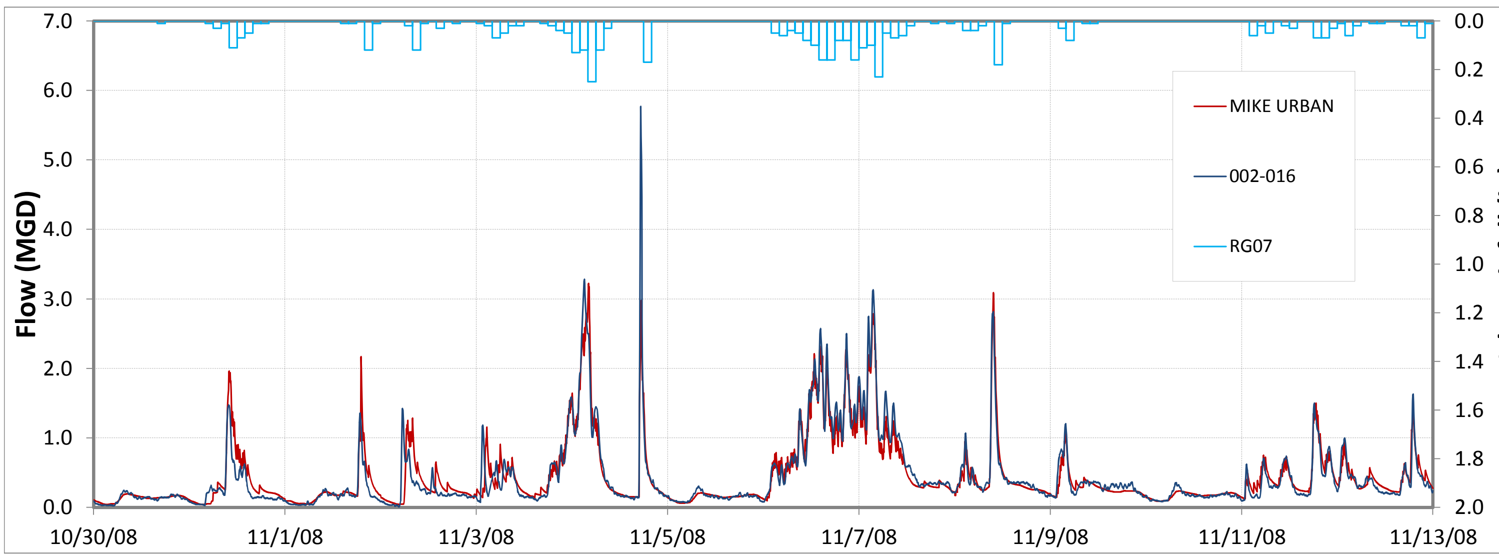
Sub-basin: **SB001a** Flow Meter: **002-016**



Event 01
09/11/08 to 09/19/08

Parameters	
rdii_area	49.1 %
b_a_iflat	30.0 %
b_m_iflat	0.079
umax	3.40 in
lmax	7.99 in
cqof	0.63
ck	6.1 hr
ckif	812.3 hr
ckbf	1810.1 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

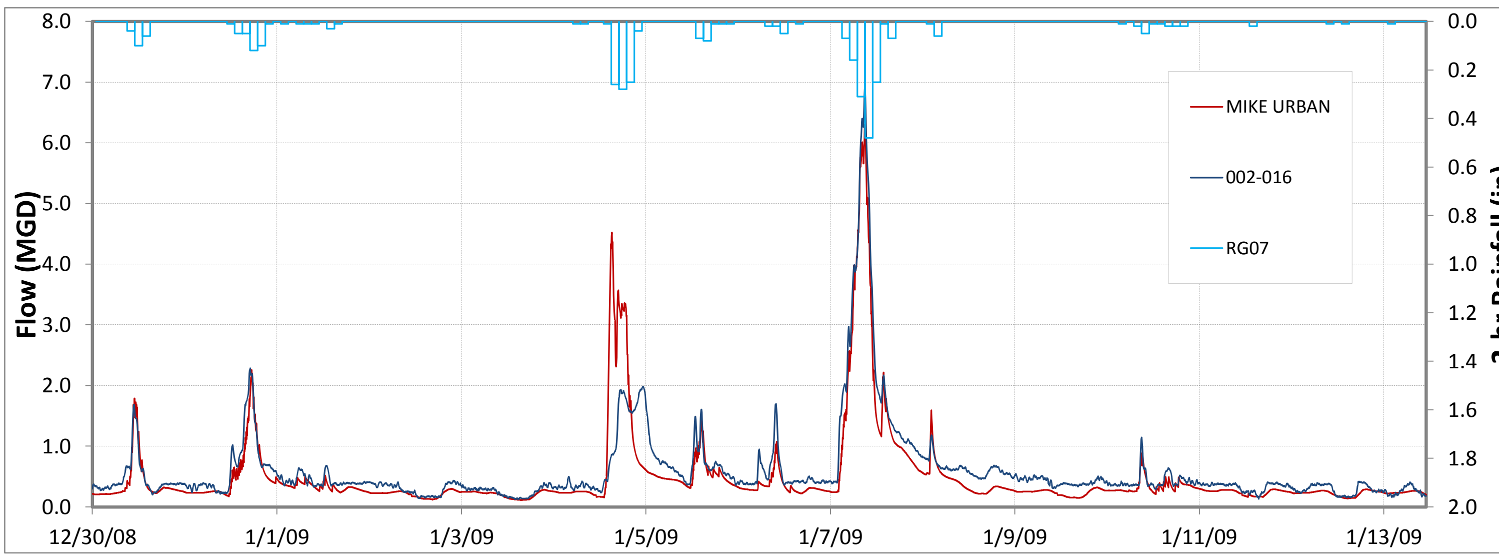
Goodness-of-Fit	
Bias	0.05
RMSE	0.02
Nash	0.76
Q_{pk} Error	-18.78%
V_{tot} Error	2.33%



Event 02
10/30/08 to 11/13/08

Parameters	
rdii_area	49.1 %
b_a_iflat	30.0 %
b_m_iflat	0.079
umax	3.40 in
lmax	7.99 in
cqof	0.63
ck	6.1 hr
ckif	812.3 hr
ckbf	1810.1 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

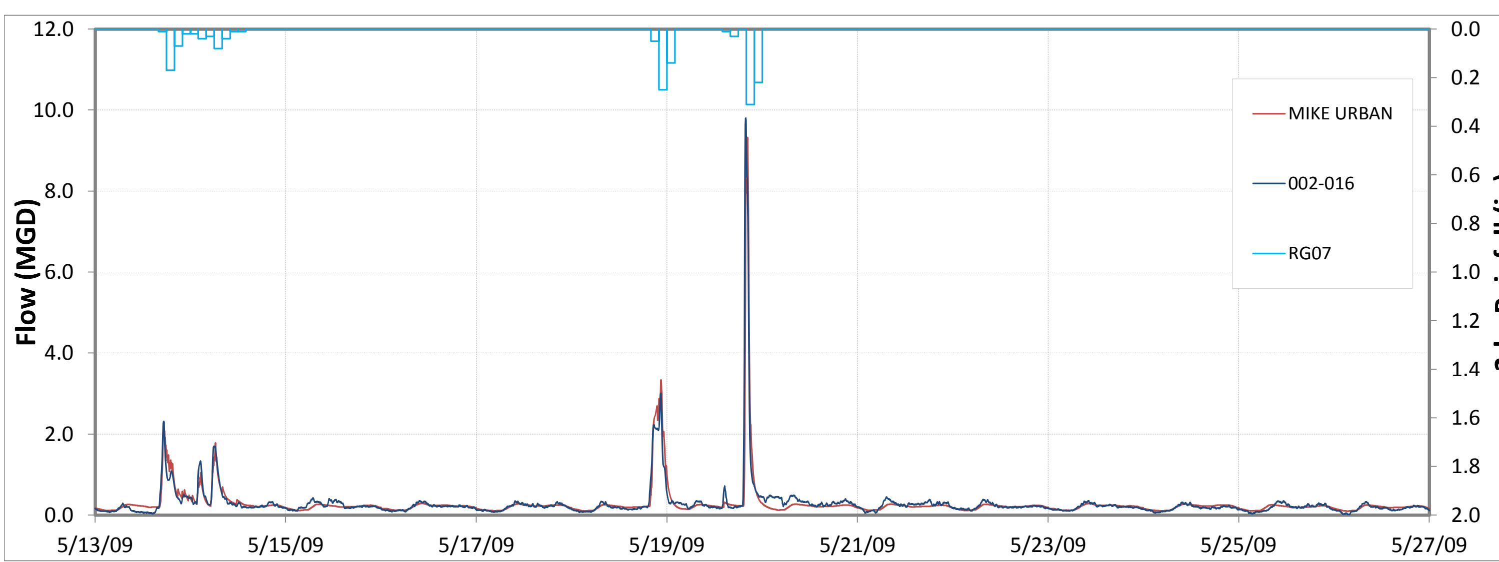
Goodness-of-Fit	
Bias	0.01
RMSE	0.18
Nash	0.88
Q_{pk} Error	-44.14%
V_{tot} Error	-4.07%



Event 03
12/30/08 to 01/14/09

Parameters	
rdii_area	49.1 %
b_a_iflat	30.0 %
b_m_iflat	0.079
umax	3.40 in
lmax	7.99 in
cqof	0.63
ck	6.1 hr
ckif	812.3 hr
ckbf	1810.1 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

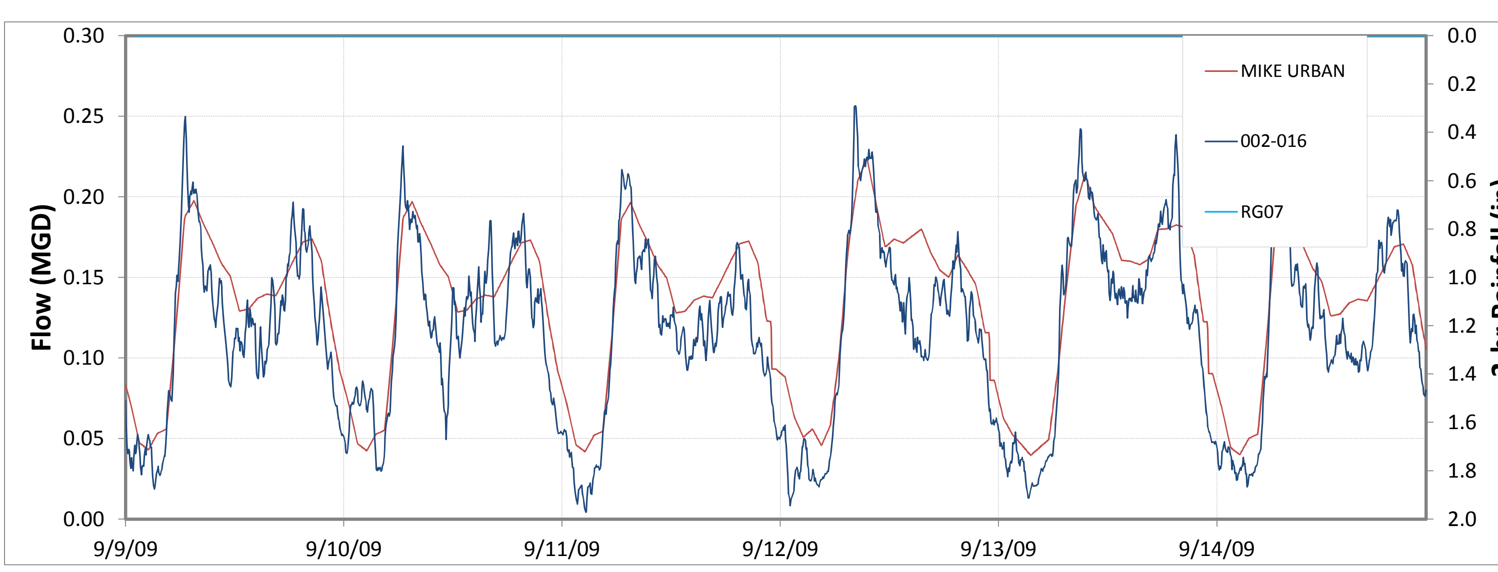
Goodness-of-Fit	
Bias	-0.18
RMSE	0.33
Nash	0.78
Q_{pk} Error	-8.51%
V_{tot} Error	-17.22%



Event 04
05/13/09 to 05/27/09

Parameters	
rdii_area	49.1 %
b_a_iflat	30.0 %
b_m_iflat	0.079
umax	3.40 in
lmax	7.99 in
cqof	0.63
ck	6.1 hr
ckif	812.3 hr
ckbf	1810.1 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

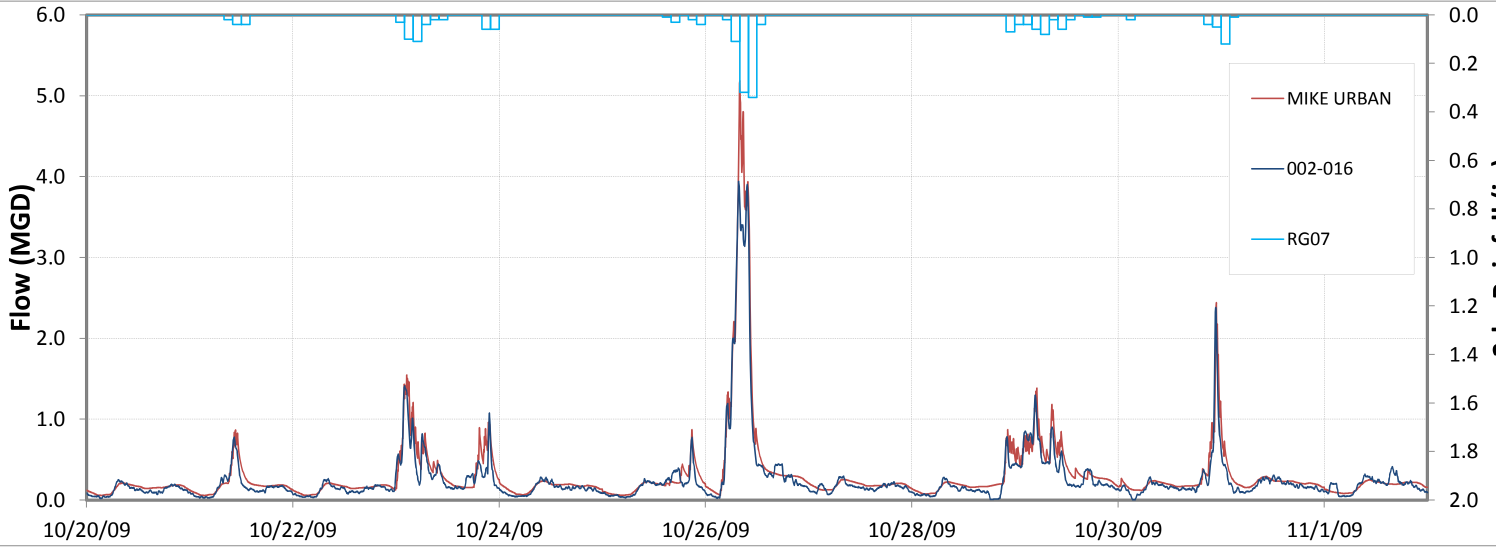
Goodness-of-Fit	
Bias	-0.02
RMSE	0.19
Nash	0.88
Q_{pk} Error	-4.89%
V_{tot} Error	-7.34%



Event 05
09/09/09 to 09/15/09

Parameters	
rdii_area	49.1 %
b_a_iflat	30.0 %
b_m_iflat	0.079
umax	3.40 in
lmax	7.99 in
cqof	0.63
ck	6.1 hr
ckif	812.3 hr
ckbf	1810.1 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.14
RMSE	0.03
Nash	0.74
Q_{pk} Error	-12.74%
V_{tot} Error	18.32%

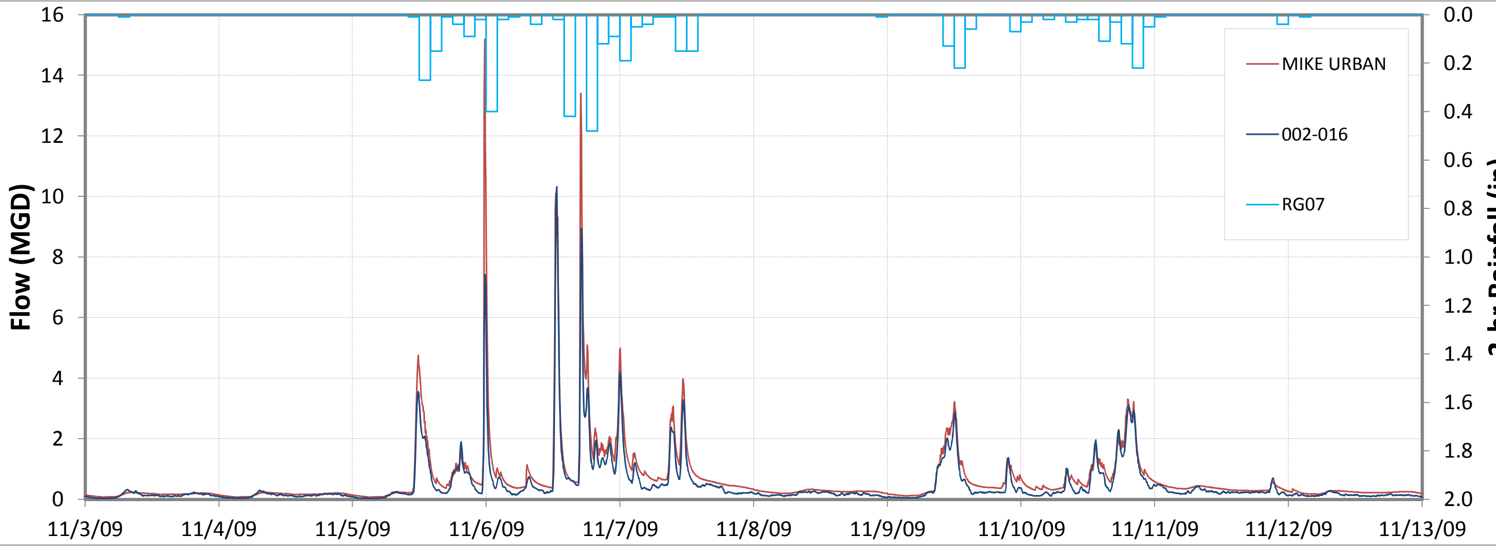


Event 06

10/20/09 to 11/02/09

Parameters	
rdii_area	49.1 %
b_a_iflat	30.0 %
b_m_iflat	0.079
umax	3.40 in
lmax	7.99 in
cqof	0.63
ck	6.1 hr
ckif	812.3 hr
ckbf	1810.1 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.18
RMSE	0.13
Nash	0.90
Q_{pk} Error	31.64%
V_{tot} Error	14.16%

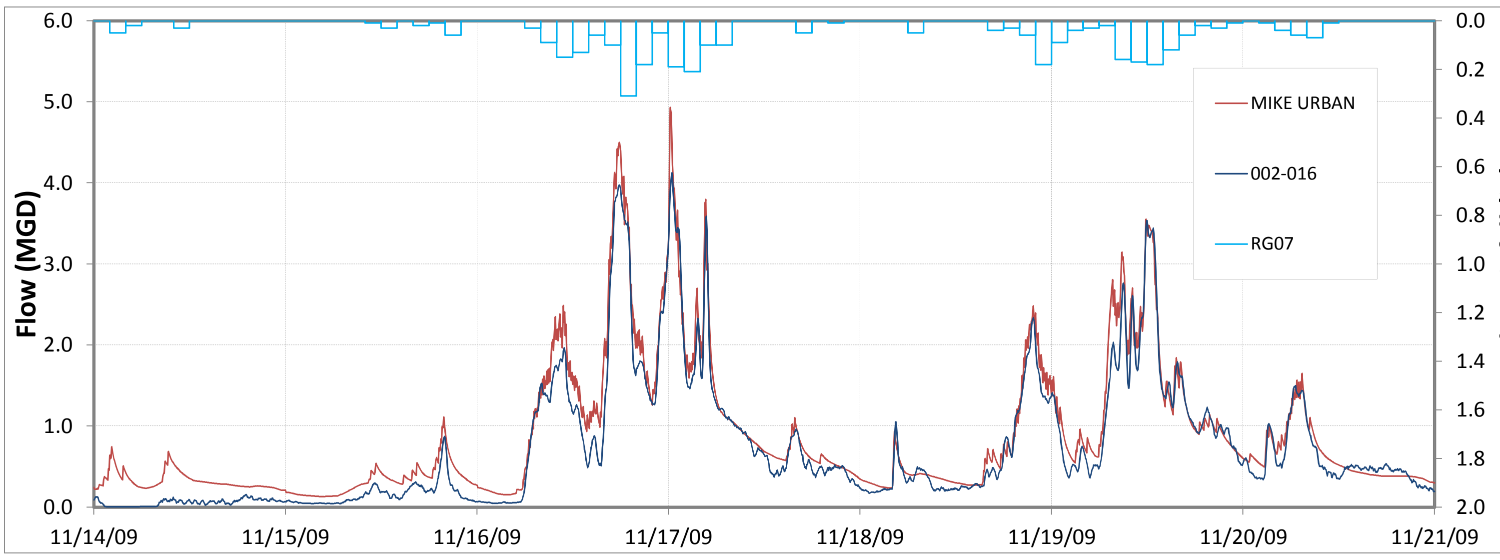


Event 07

11/03/09 to 11/13/09

Parameters	
rdii_area	49.1 %
b_a_iflat	30.0 %
b_m_iflat	0.079
umax	3.40 in
lmax	7.99 in
cqof	0.63
ck	6.1 hr
ckif	812.3 hr
ckbf	1810.1 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.36
RMSE	0.40
Nash	0.77
Q_{pk} Error	47.18%
V_{tot} Error	34.45%

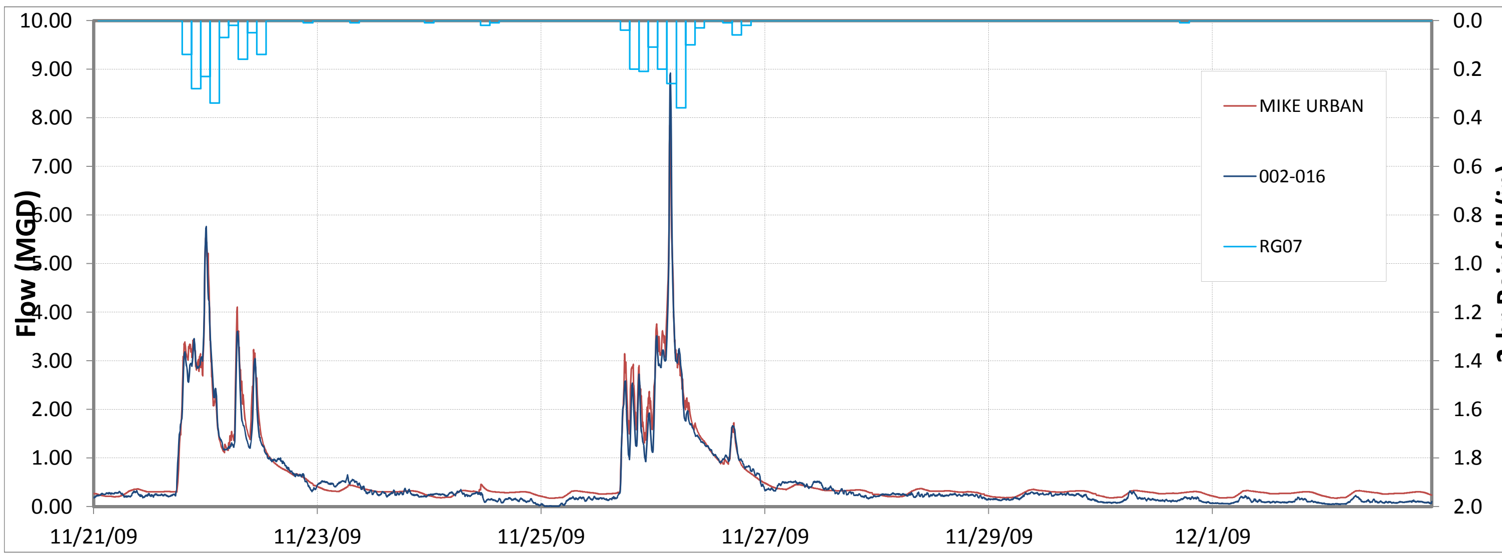


Event 08

11/14/09 to 11/21/09

Parameters	
rdii_area	49.1 %
b_a_iflat	30.0 %
b_m_iflat	0.079
umax	3.40 in
lmax	7.99 in
cqof	0.63
ck	6.1 hr
ckif	812.3 hr
ckbf	1810.1 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.20
RMSE	0.22
Nash	0.93
Q_{pk} Error	19.46%
V_{tot} Error	14.32%

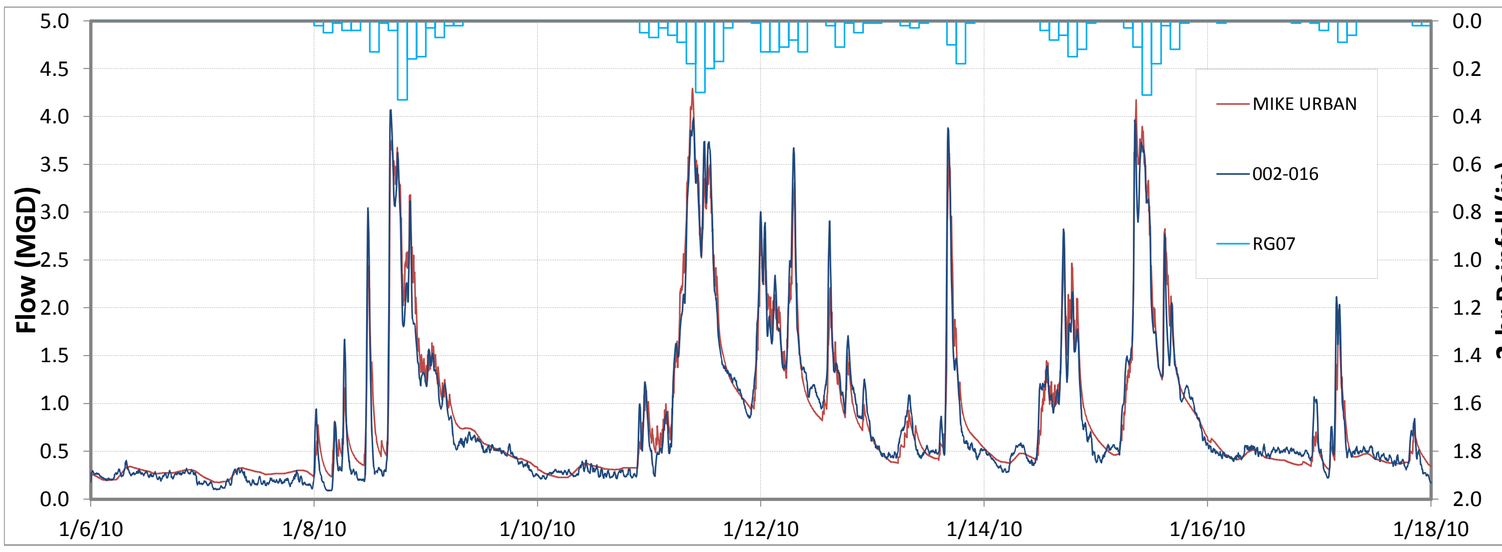


Event 09

11/21/09 to 12/03/09

Parameters	
rdii_area	49.1 %
b_a_iflat	30.0 %
b_m_iflat	0.079
umax	3.40 in
lmax	7.99 in
cqof	0.63
ck	6.1 hr
ckif	812.3 hr
ckbf	1810.1 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.15
RMSE	0.16
Nash	0.96
Q_{pk} Error	-11.89%
V_{tot} Error	8.95%



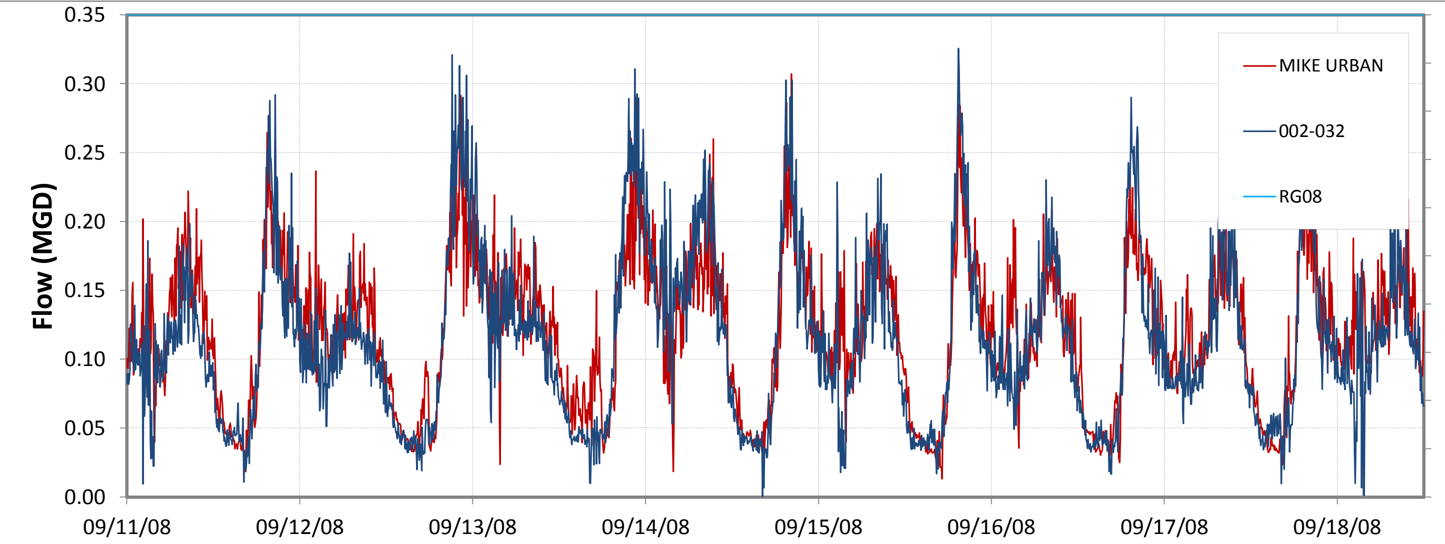
Event 10

01/06/10 to 01/18/10

Parameters	
rdii_area	49.1 %
b_a_iflat	30.0 %
b_m_iflat	0.079
umax	3.40 in
lmax	7.99 in
cqof	0.63
ck	6.1 hr
ckif	812.3 hr
ckbf	1810.1 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.03
RMSE	0.16
Nash	0.96
Q_{pk} Error	5.51%
V_{tot} Error	2.26%

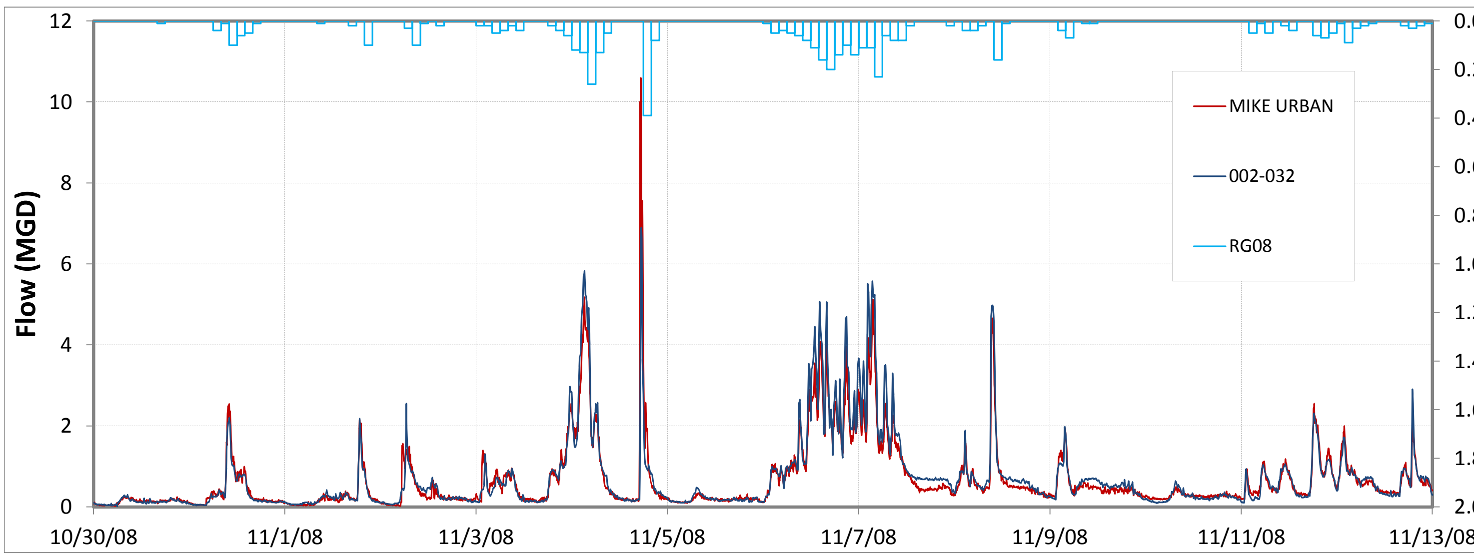
Sub-basin: **SB001b** Flow Meter: **002-032**



Event 01
09/11/08 to 09/19/08

Parameters	
rdii_area	78.0 %
b_a_iflat	14.9 %
b_m_iflat	0.064
umax	3.93 in
lmax	6.00 in
cqof	0.33
ck	3.3 hr
ckif	458.3 hr
ckbf	853.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

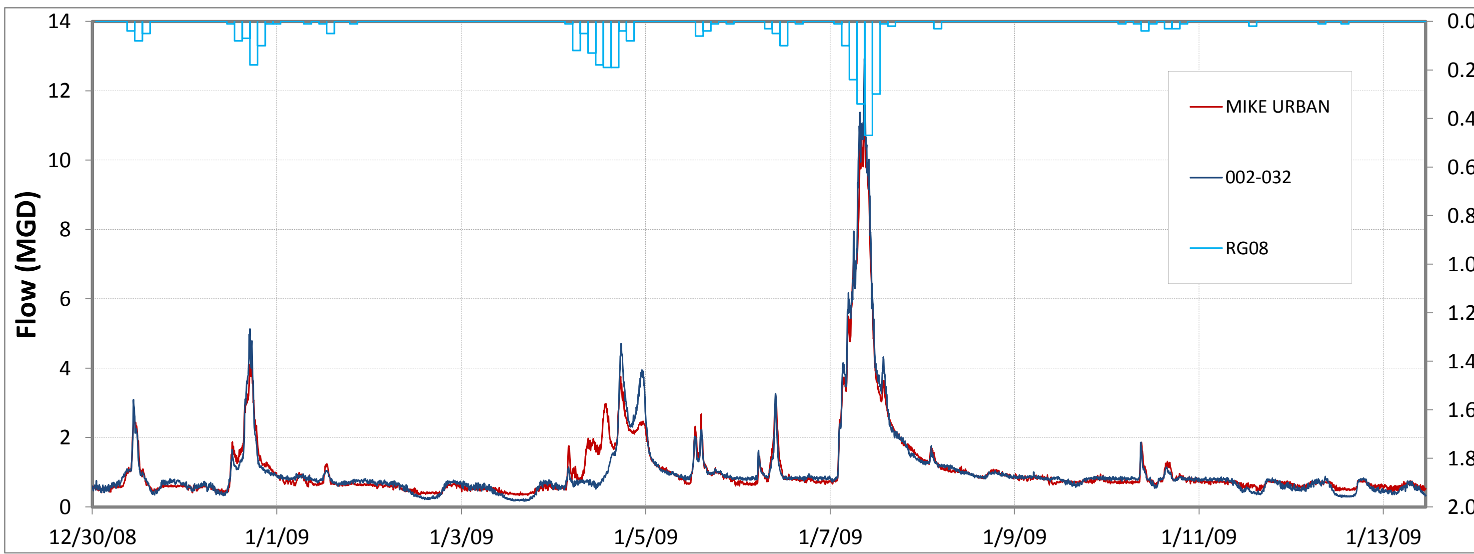
Goodness-of-Fit	
Bias	0.06
RMSE	0.03
Nash	0.67
Q_{pk} Error	-4.97%
V_{tot} Error	7.45%



Event 02
10/30/08 to 11/13/08

Parameters	
rdii_area	78.0 %
b_a_iflat	14.9 %
b_m_iflat	0.064
umax	3.93 in
lmax	6.00 in
cqof	0.33
ck	3.3 hr
ckif	458.3 hr
ckbf	853.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

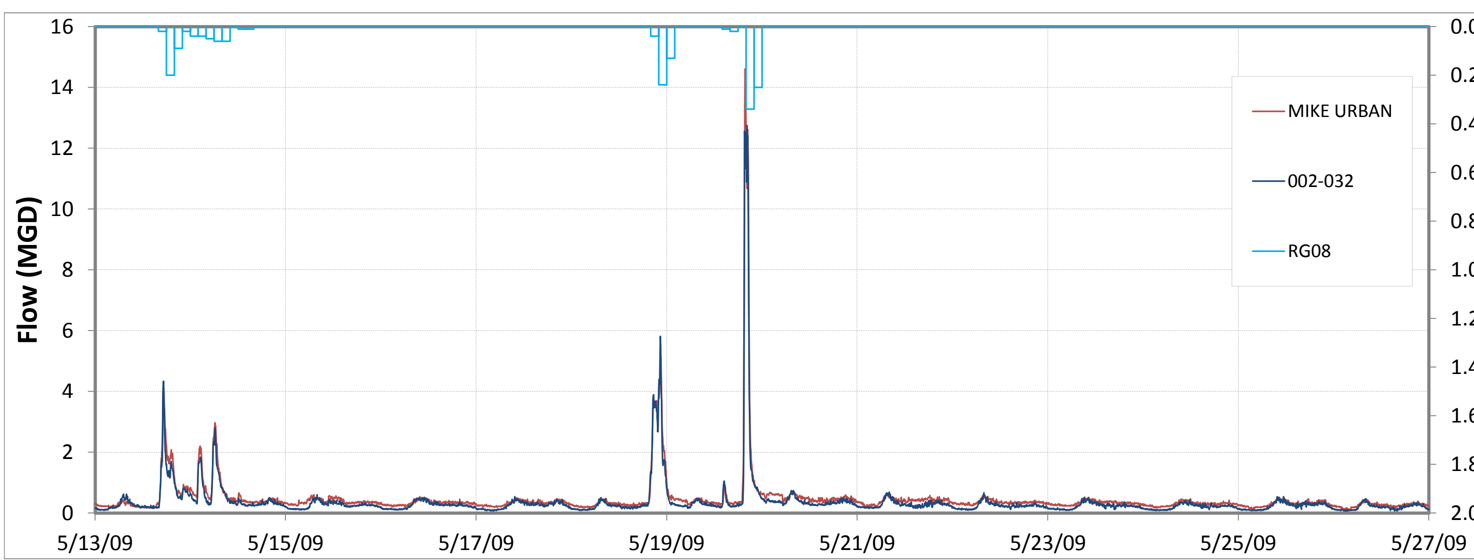
Goodness-of-Fit	
Bias	-0.06
RMSE	0.31
Nash	0.89
Q_{pk} Error	53.79%
V_{tot} Error	1.28%



Event 03
12/30/08 to 01/14/09

Parameters	
rdii_area	78.0 %
b_a_iflat	14.9 %
b_m_iflat	0.064
umax	3.93 in
lmax	6.00 in
cqof	0.33
ck	3.3 hr
ckif	458.3 hr
ckbf	853.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

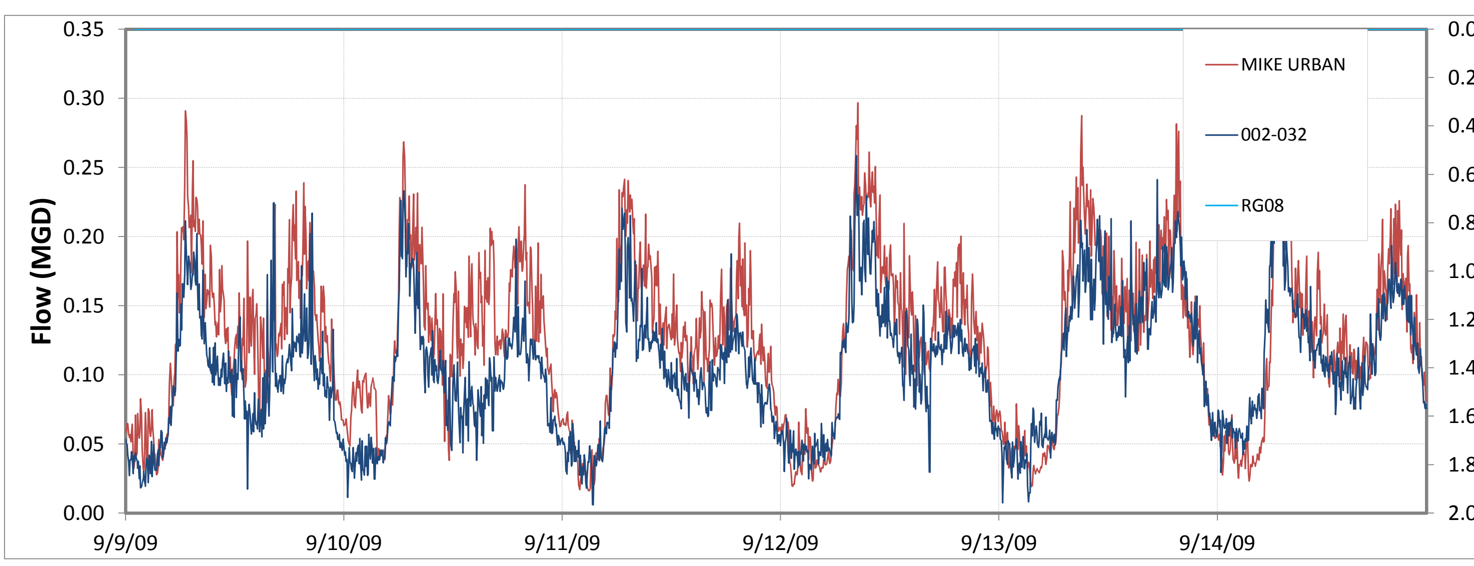
Goodness-of-Fit	
Bias	0.00
RMSE	0.27
Nash	0.95
Q_{pk} Error	-14.26%
V_{tot} Error	-4.40%



Event 04
05/13/09 to 05/27/09

Parameters	
rdii_area	78.0 %
b_a_iflat	14.9 %
b_m_iflat	0.064
umax	3.93 in
lmax	6.00 in
cqof	0.33
ck	3.3 hr
ckif	458.3 hr
ckbf	853.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

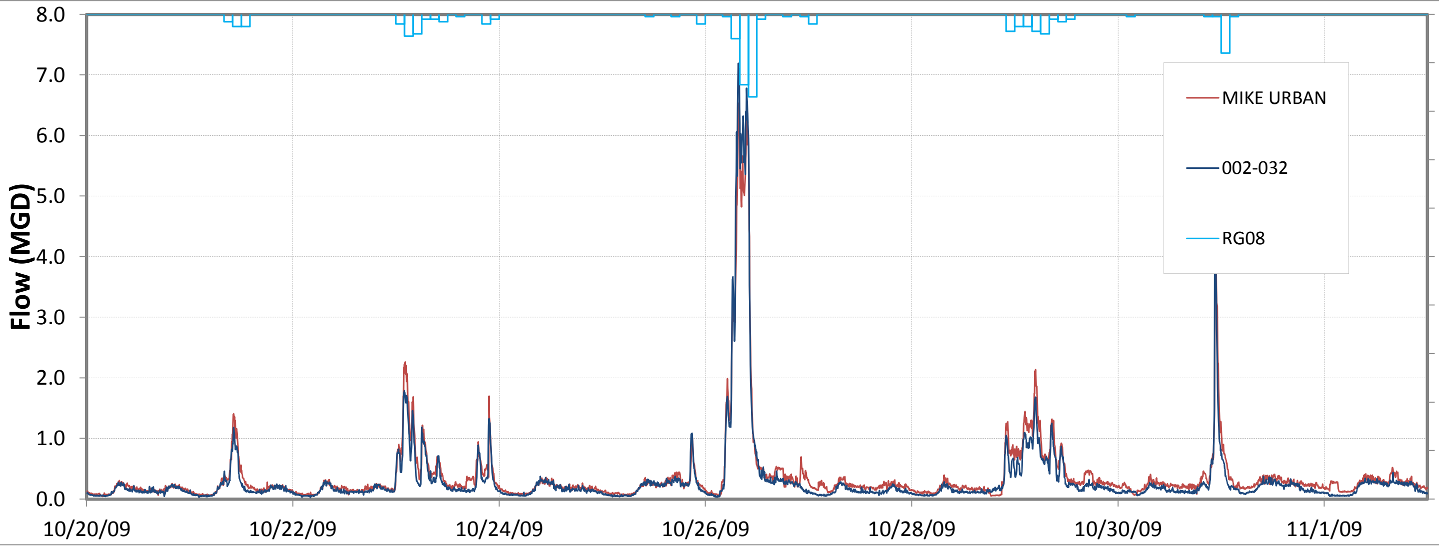
Goodness-of-Fit	
Bias	0.26
RMSE	0.17
Nash	0.95
Q_{pk} Error	14.59%
V_{tot} Error	15.81%



Event 05
09/09/09 to 09/15/09

Parameters	
rdii_area	78.0 %
b_a_iflat	14.9 %
b_m_iflat	0.064
umax	3.93 in
lmax	6.00 in
cqof	0.33
ck	3.3 hr
ckif	458.3 hr
ckbf	853.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

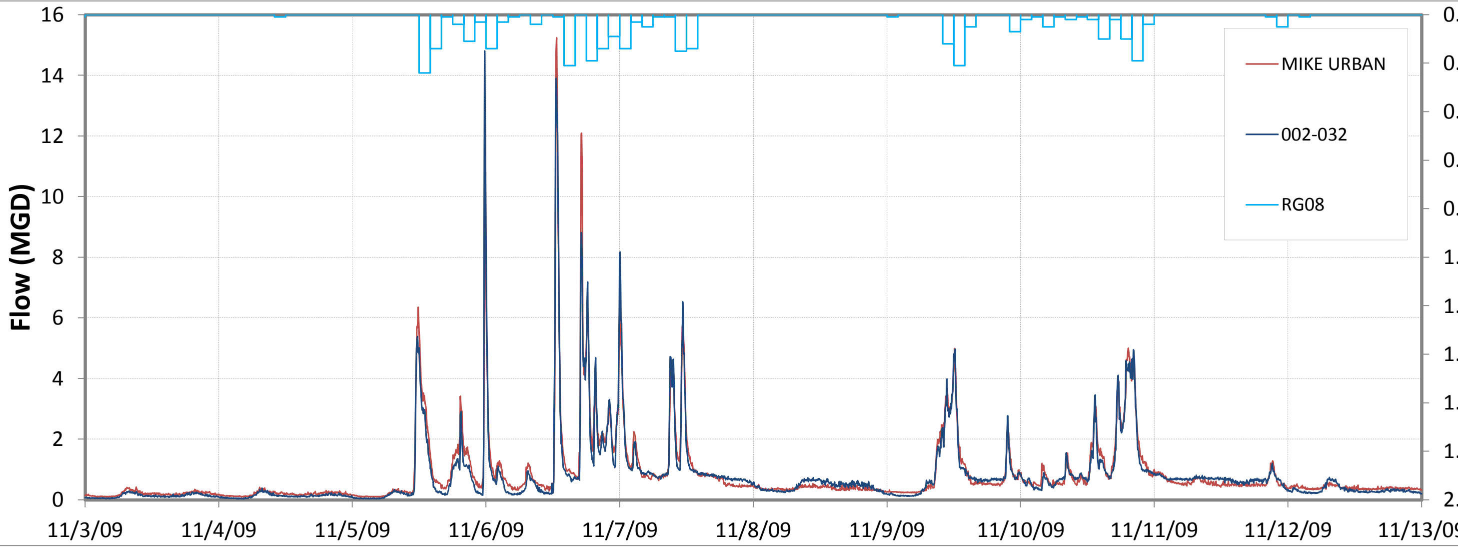
Goodness-of-Fit	
Bias	0.21
RMSE	0.04
Nash	0.38
Q_{pk} Error	15.40%
V_{tot} Error	7.43%



Event 06
10/20/09 to 11/02/09

Parameters	
rdii_area	78.0 %
b_a_iflat	14.9 %
b_m_iflat	0.064
umax	3.93 in
lmax	6.00 in
cqof	0.33
ck	3.3 hr
ckif	458.3 hr
ckbf	853.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

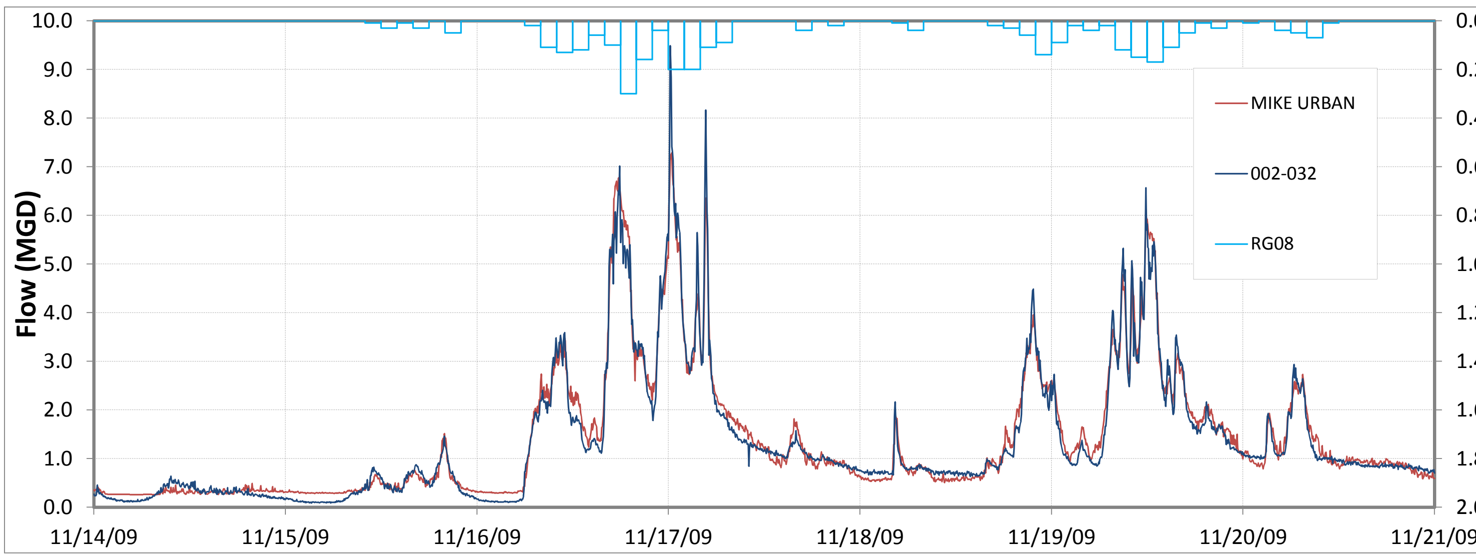
Goodness-of-Fit	
Bias	0.19
RMSE	0.14
Nash	0.96
Q_{pk} Error	-9.20%
V_{tot} Error	1.96%



Event 07
11/03/09 to 11/13/09

Parameters	
rdii_area	78.0 %
b_a_iflat	14.9 %
b_m_iflat	0.064
umax	3.93 in
lmax	6.00 in
cqof	0.33
ck	3.3 hr
ckif	458.3 hr
ckbf	853.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

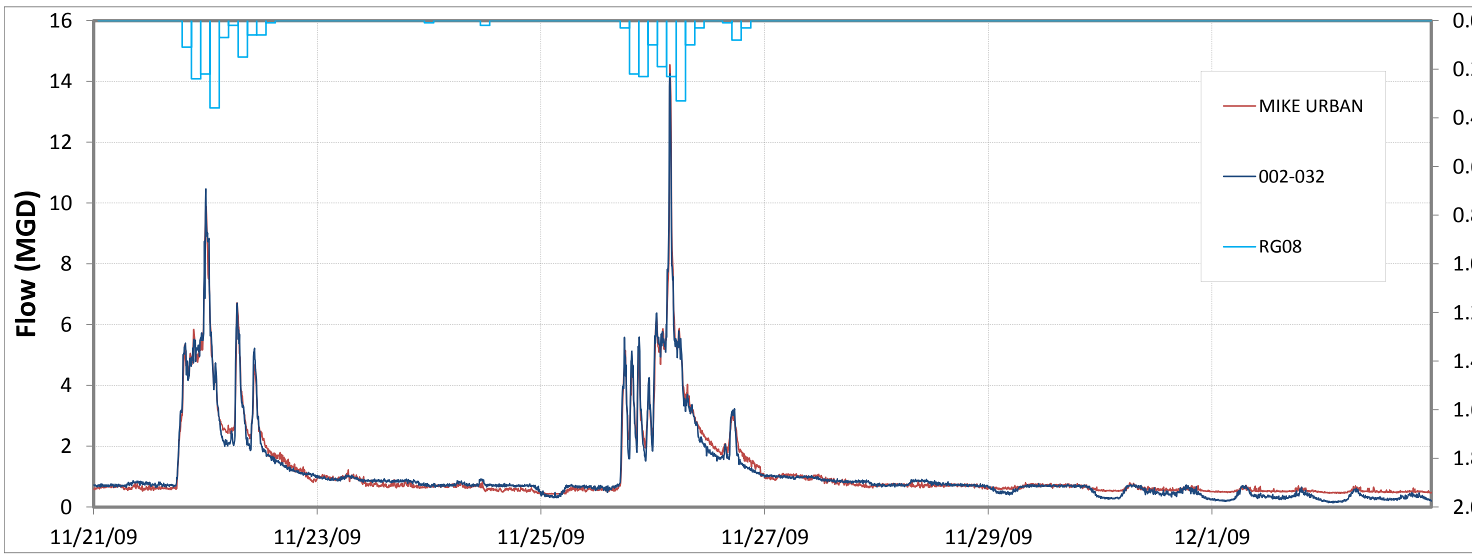
Goodness-of-Fit	
Bias	0.06
RMSE	0.26
Nash	0.95
Q_{pk} Error	2.91%
V_{tot} Error	9.96%



Event 08
11/14/09 to 11/21/09

Parameters	
rdii_area	78.0 %
b_a_iflat	14.9 %
b_m_iflat	0.064
umax	3.93 in
lmax	6.00 in
cqof	0.33
ck	3.3 hr
ckif	458.3 hr
ckbf	853.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

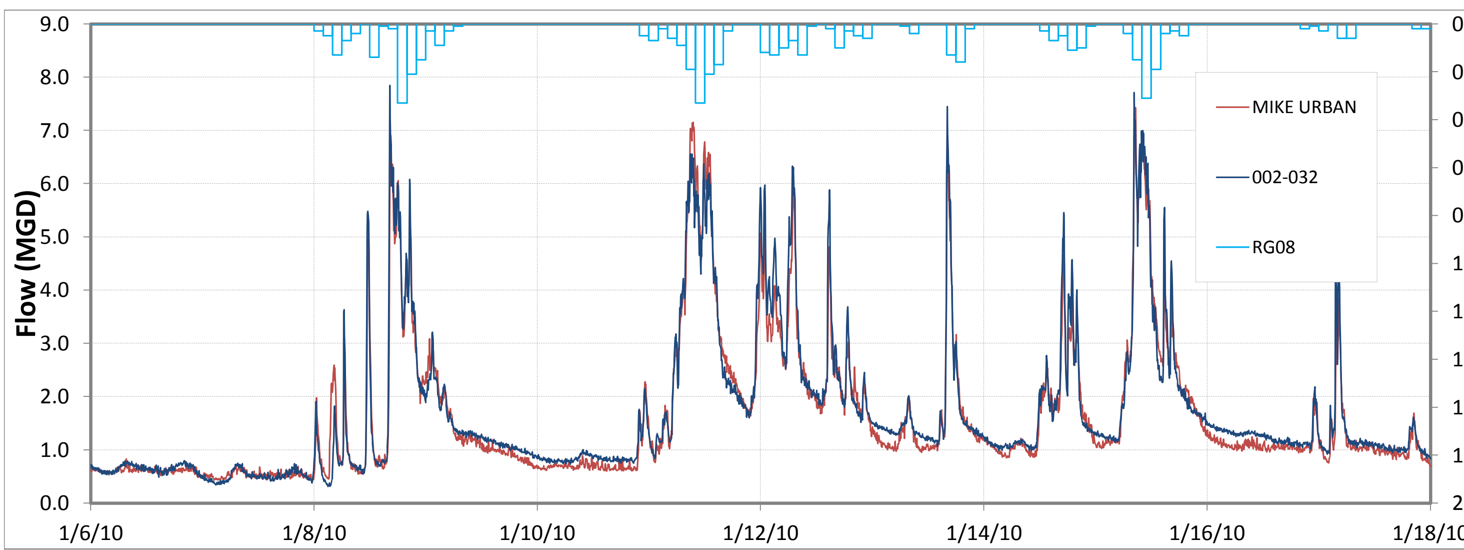
Goodness-of-Fit	
Bias	0.03
RMSE	0.23
Nash	0.97
Q_{pk} Error	-23.21%
V_{tot} Error	3.07%



Event 09
11/21/09 to 12/03/09

Parameters	
rdii_area	78.0 %
b_a_iflat	14.9 %
b_m_iflat	0.064
umax	3.93 in
lmax	6.00 in
cqof	0.33
ck	3.3 hr
ckif	458.3 hr
ckbf	853.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.03
RMSE	0.20
Nash	0.98
Q_{pk} Error	2.06%
V_{tot} Error	2.39%

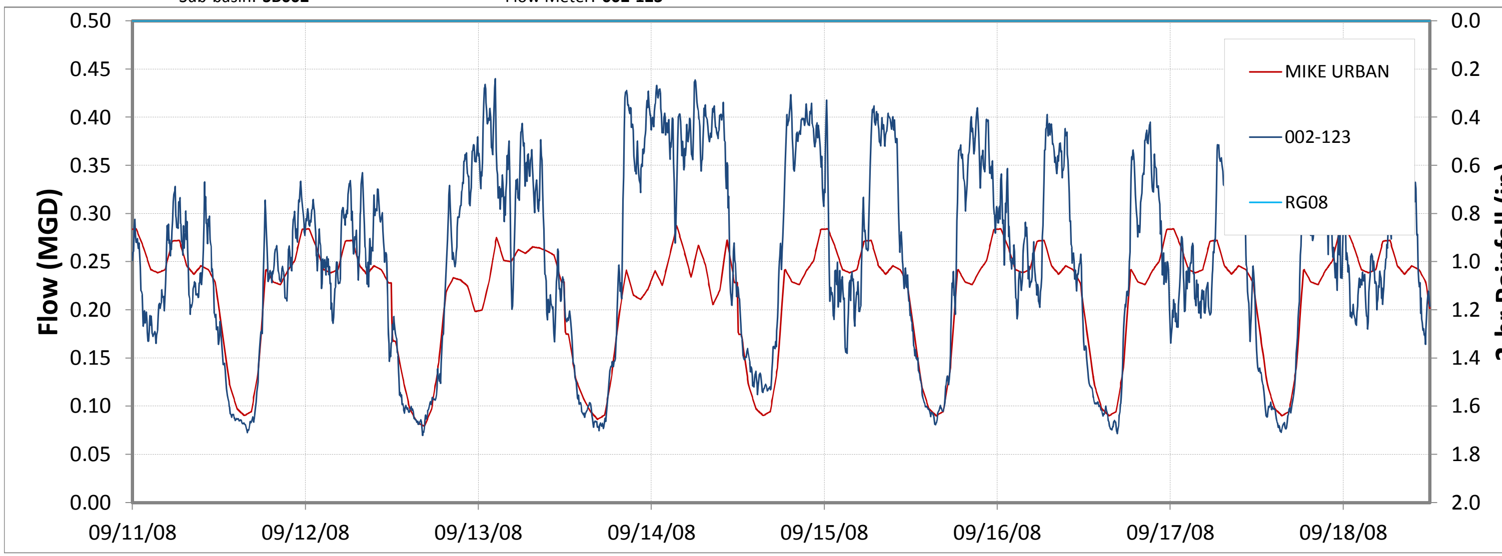


Event 10
01/06/10 to 01/18/10

Parameters	
rdii_area	78.0 %
b_a_iflat	14.9 %
b_m_iflat	0.064
umax	3.93 in
lmax	6.00 in
cqof	0.33
ck	3.3 hr
ckif	458.3 hr
ckbf	853.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	-0.04
RMSE	0.28
Nash	0.96
Q_{pk} Error	-5.29%
V_{tot} Error	-3.15%

Sub-basin: **SB002** Flow Meter: **002-123**

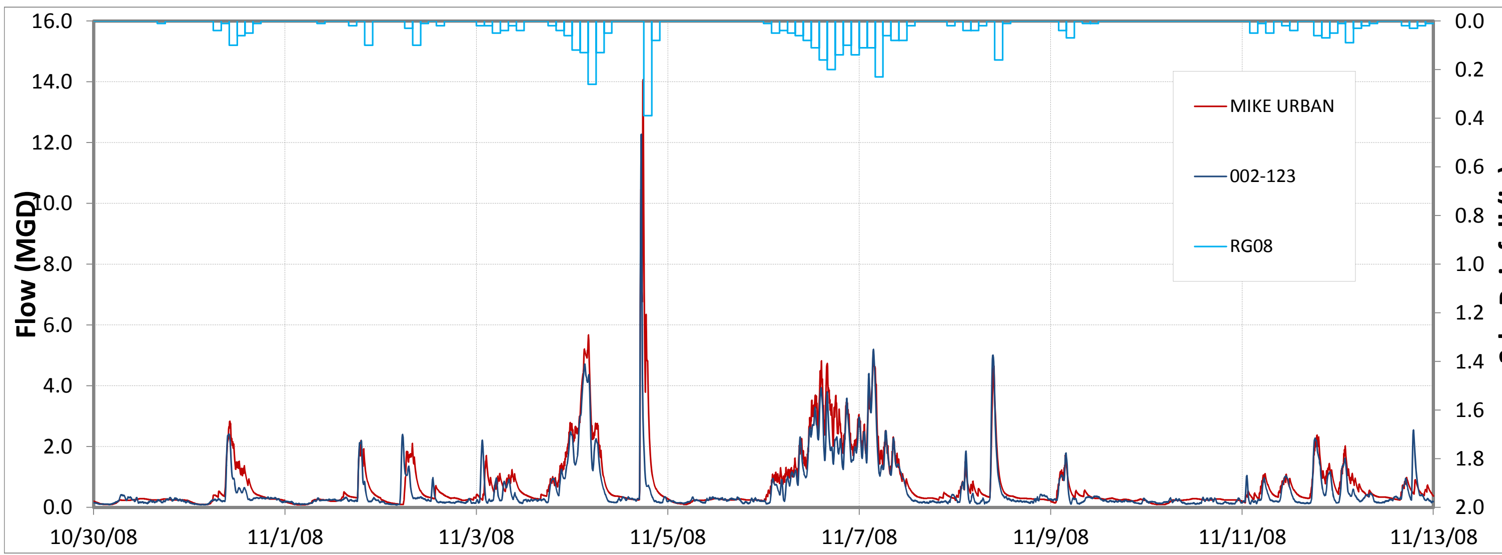


Event 01

09/11/08 to 09/19/08

Parameters	
rdii_area	16.0 %
b_a_iflat	25.0 %
b_m_iflat	0.061
umax	3.94 in
lmax	4.14 in
cqof	0.76
ck	6.2 hr
ckif	898.1 hr
ckbf	1000.0 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	-0.15
RMSE	0.08
Nash	0.39
Q_{pk} Error	-34.73%
V_{tot} Error	-24.62%

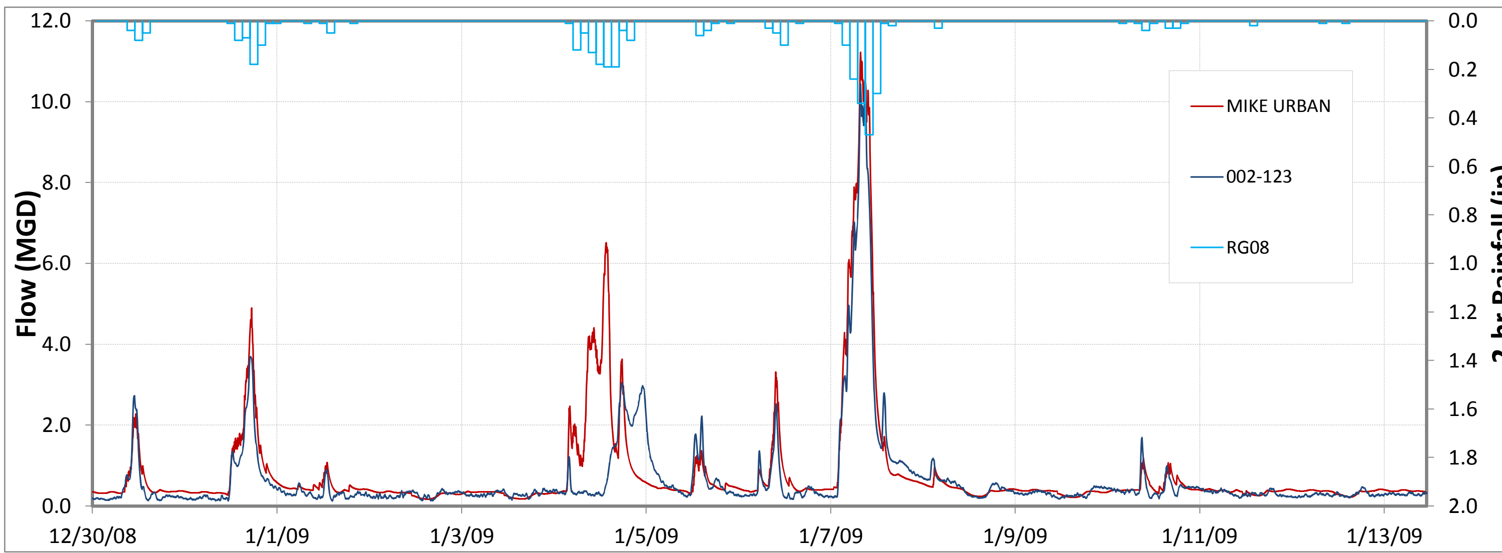


Event 02

10/30/08 to 11/13/08

Parameters	
rdii_area	16.0 %
b_a_iflat	25.0 %
b_m_iflat	0.061
umax	3.94 in
lmax	4.14 in
cqof	0.76
ck	6.2 hr
ckif	898.1 hr
ckbf	1000.0 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.29
RMSE	0.53
Nash	0.61
Q_{pk} Error	14.69%
V_{tot} Error	41.67%

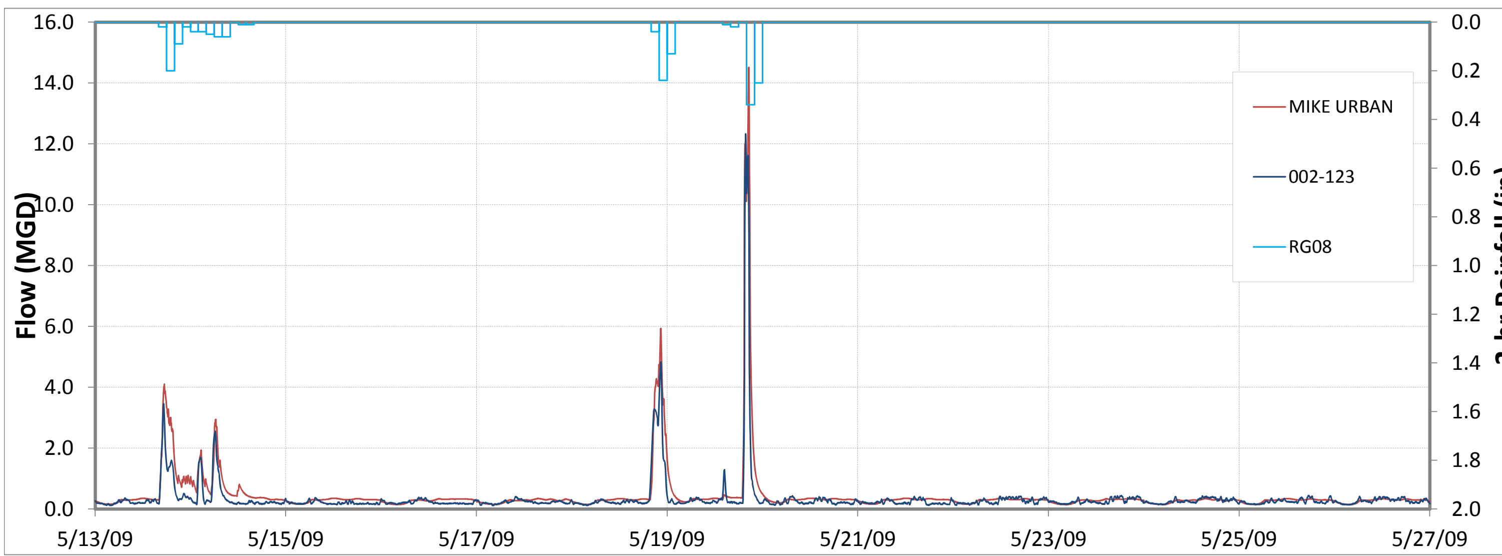


Event 03

12/30/08 to 01/14/09

Parameters	
rdii_area	16.0 %
b_a_iflat	25.0 %
b_m_iflat	0.061
umax	3.94 in
lmax	4.14 in
cqof	0.76
ck	6.2 hr
ckif	898.1 hr
ckbf	1000.0 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.22
RMSE	0.66
Nash	0.64
Q_{pk} Error	6.65%
V_{tot} Error	11.92%

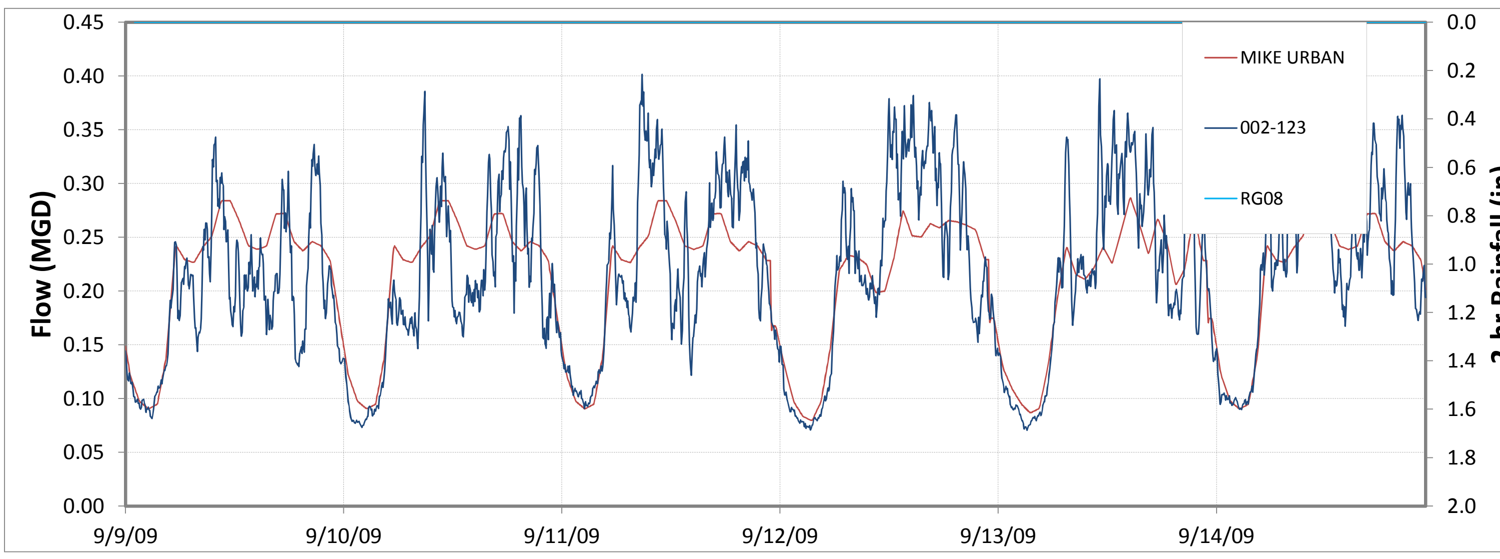


Event 04

05/13/09 to 05/27/09

Parameters	
rdii_area	16.0 %
b_a_iflat	25.0 %
b_m_iflat	0.061
umax	3.94 in
lmax	4.14 in
cqof	0.76
ck	6.2 hr
ckif	898.1 hr
ckbf	1000.0 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.30
RMSE	0.35
Nash	0.77
Q_{pk} Error	17.84%
V_{tot} Error	38.83%



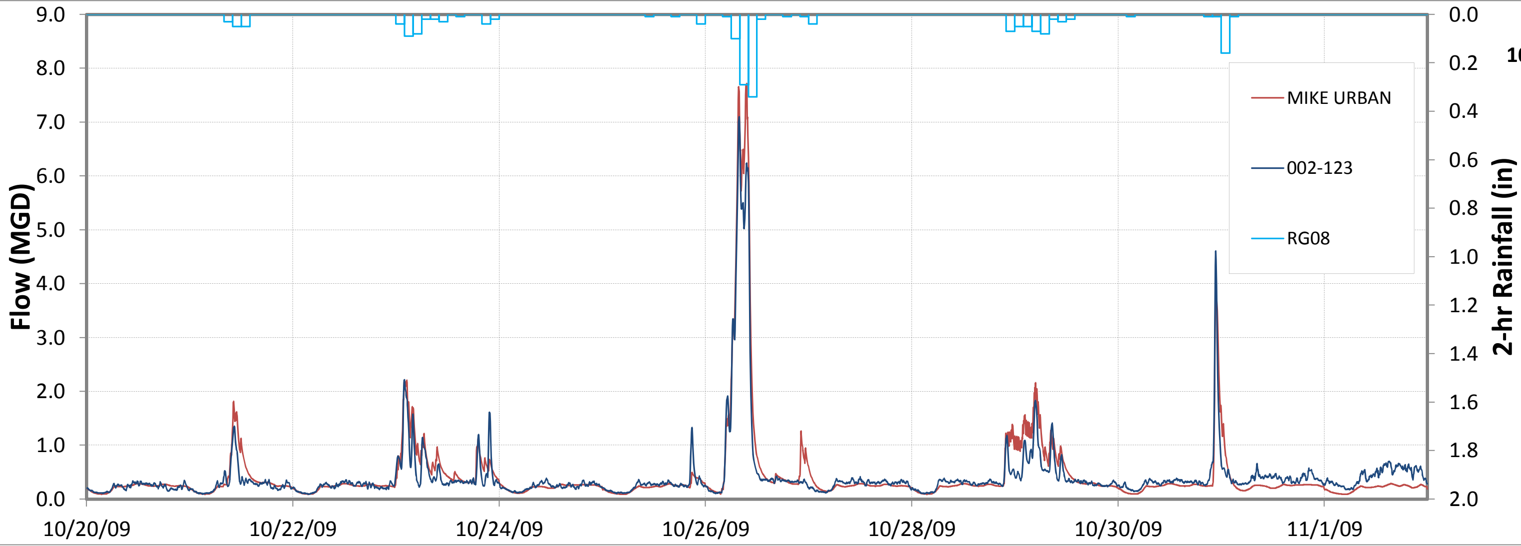
Event 05

09/09/09 to 09/15/09

Parameters	
rdii_area	16.0 %
b_a_iflat	25.0 %
b_m_iflat	0.061
umax	3.94 in
lmax	4.14 in
cqof	0.76
ck	6.2 hr
ckif	898.1 hr
ckbf	1000.0 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	-0.01
RMSE	0.05
Nash	0.64
Q_{pk} Error	-28.58%
V_{tot} Error	-5.61%

SB002

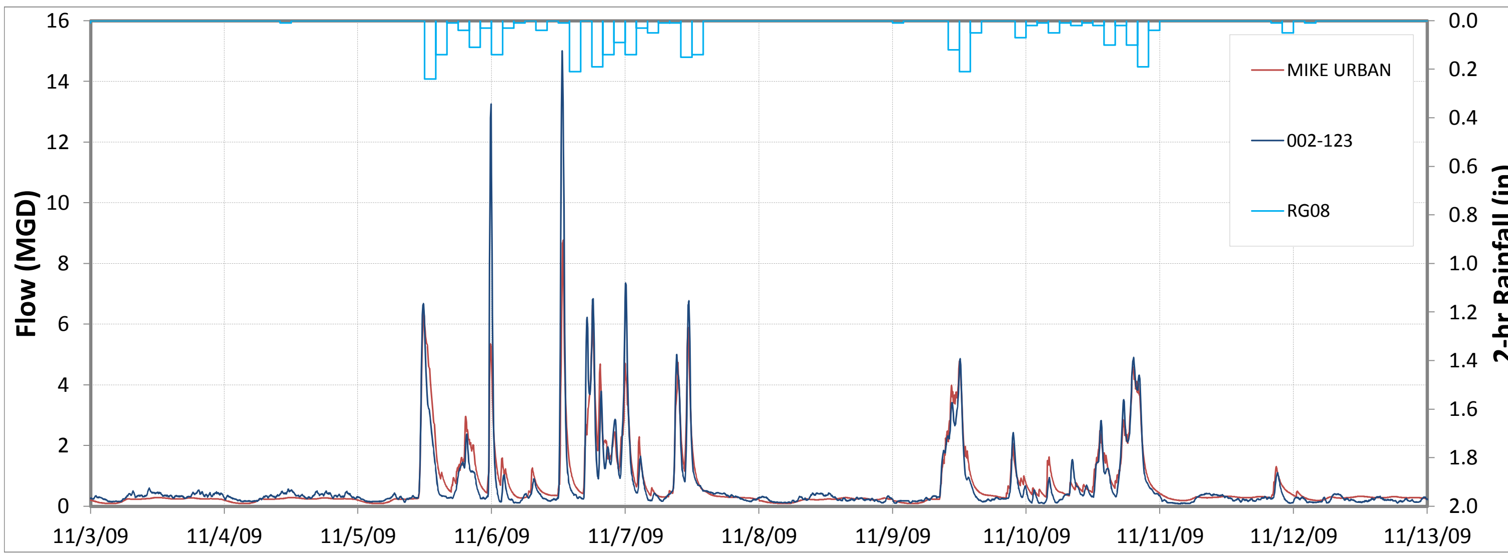


Event 06

10/20/09 to 11/02/09

Parameters	
rdii_area	16.0 %
b_a_iflat	25.0 %
b_m_iflat	0.061
umax	3.94 in
lmax	4.14 in
cqof	0.76
ck	6.2 hr
ckif	898.1 hr
ckbf	1000.0 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.02
RMSE	0.22
Nash	0.89
Q_{pk} Error	8.74%
V_{tot} Error	12.10%

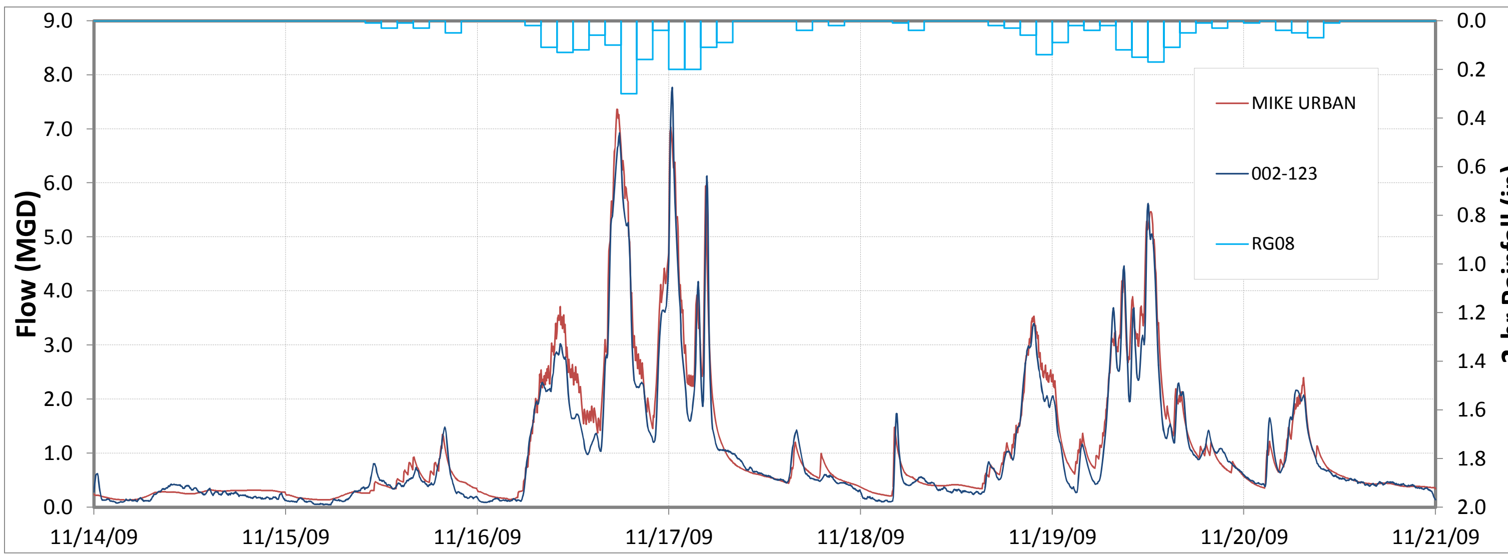


Event 07

11/03/09 to 11/13/09

Parameters	
rdii_area	16.0 %
b_a_iflat	25.0 %
b_m_iflat	0.061
umax	3.94 in
lmax	4.14 in
cqof	0.76
ck	6.2 hr
ckif	898.1 hr
ckbf	1000.0 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.03
RMSE	0.53
Nash	0.82
Q_{pk} Error	-41.59%
V_{tot} Error	3.00%

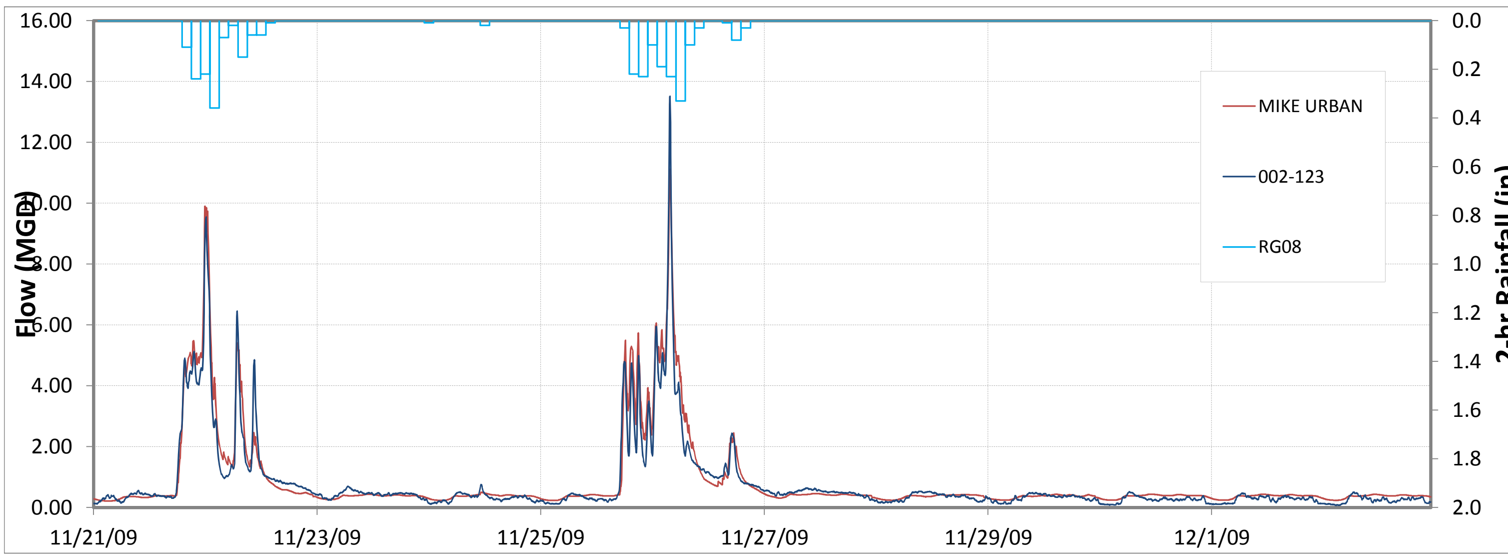


Event 08

11/14/09 to 11/21/09

Parameters	
rdii_area	16.0 %
b_a_iflat	25.0 %
b_m_iflat	0.061
umax	3.94 in
lmax	4.14 in
cqof	0.76
ck	6.2 hr
ckif	898.1 hr
ckbf	1000.0 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.10
RMSE	0.27
Nash	0.95
Q_{pk} Error	-5.18%
V_{tot} Error	12.24%

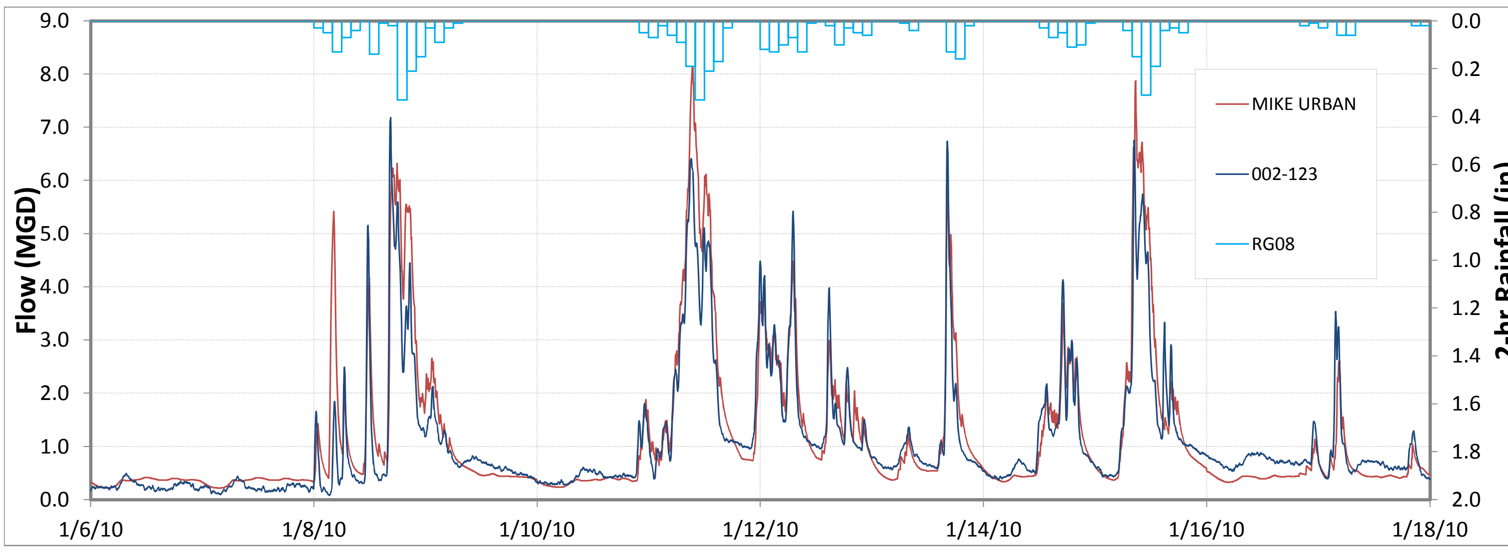


Event 09

11/21/09 to 12/03/09

Parameters	
rdii_area	16.0 %
b_a_iflat	25.0 %
b_m_iflat	0.061
umax	3.94 in
lmax	4.14 in
cqof	0.76
ck	6.2 hr
ckif	898.1 hr
ckbf	1000.0 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.09
RMSE	0.32
Nash	0.94
Q_{pk} Error	-17.73%
V_{tot} Error	10.75%



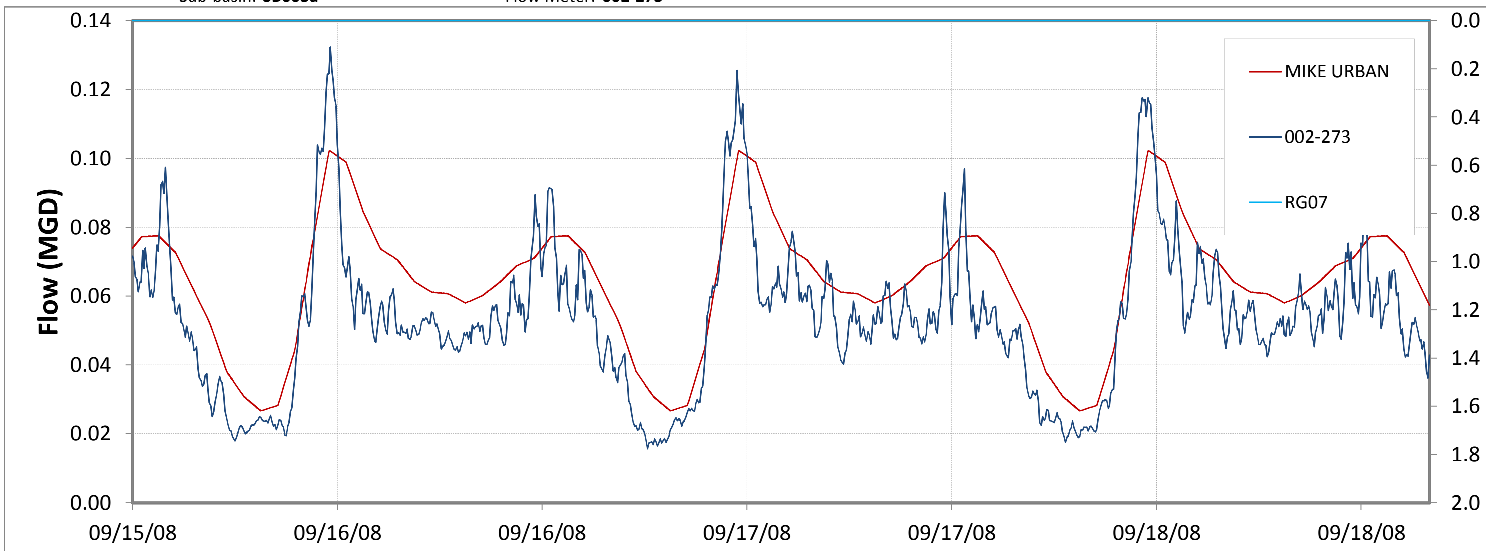
Event 10

01/06/10 to 01/18/10

Parameters	
rdii_area	16.0 %
b_a_iflat	25.0 %
b_m_iflat	0.061
umax	3.94 in
lmax	4.14 in
cqof	0.76
ck	6.2 hr
ckif	898.1 hr
ckbf	1000.0 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.07
RMSE	0.51
Nash	0.81
Q_{pk} Error	13.34%
V_{tot} Error	9.56%

Sub-basin: **SB003a** Flow Meter: **002-273**

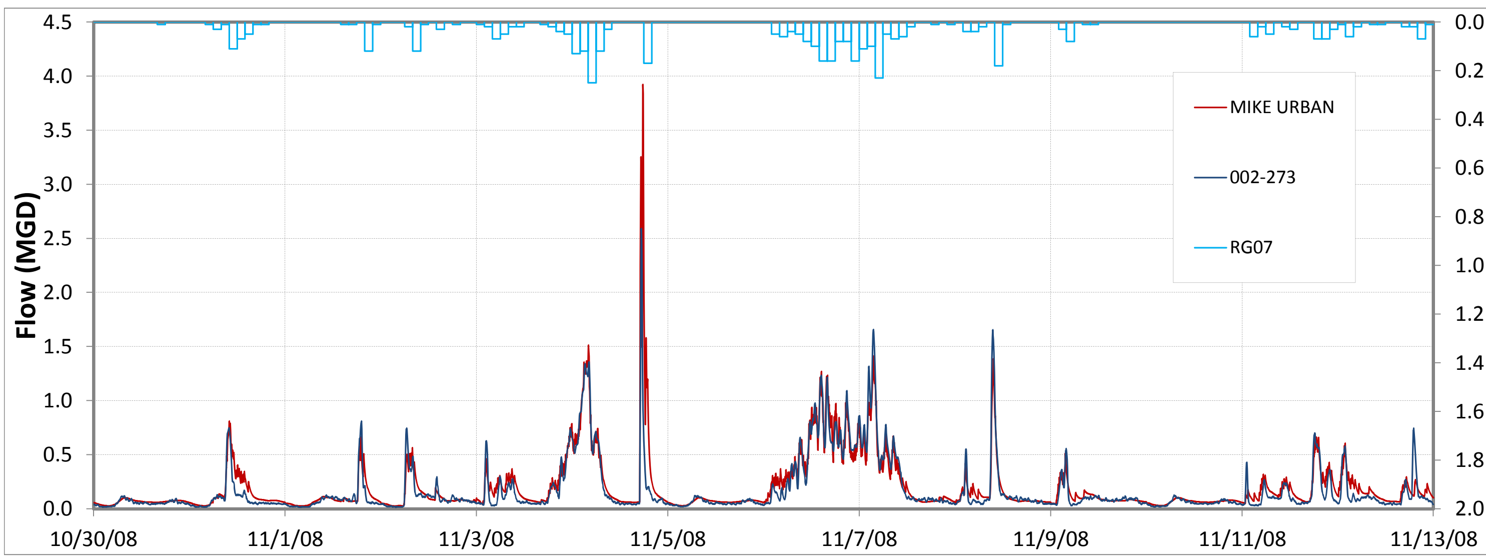


Event 01

09/15/08 to 09/19/08

Parameters	
rdii_area	7.9 %
b_a_iflat	23.0 %
b_m_iflat	0.069
umax	4.66 in
lmax	5.91 in
cqof	1.00
ck	2.7 hr
ckif	1188.1 hr
ckbf	100.0 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.19
RMSE	0.01
Nash	0.53
Q_{pk} Error	-22.78%
V_{tot} Error	18.11%

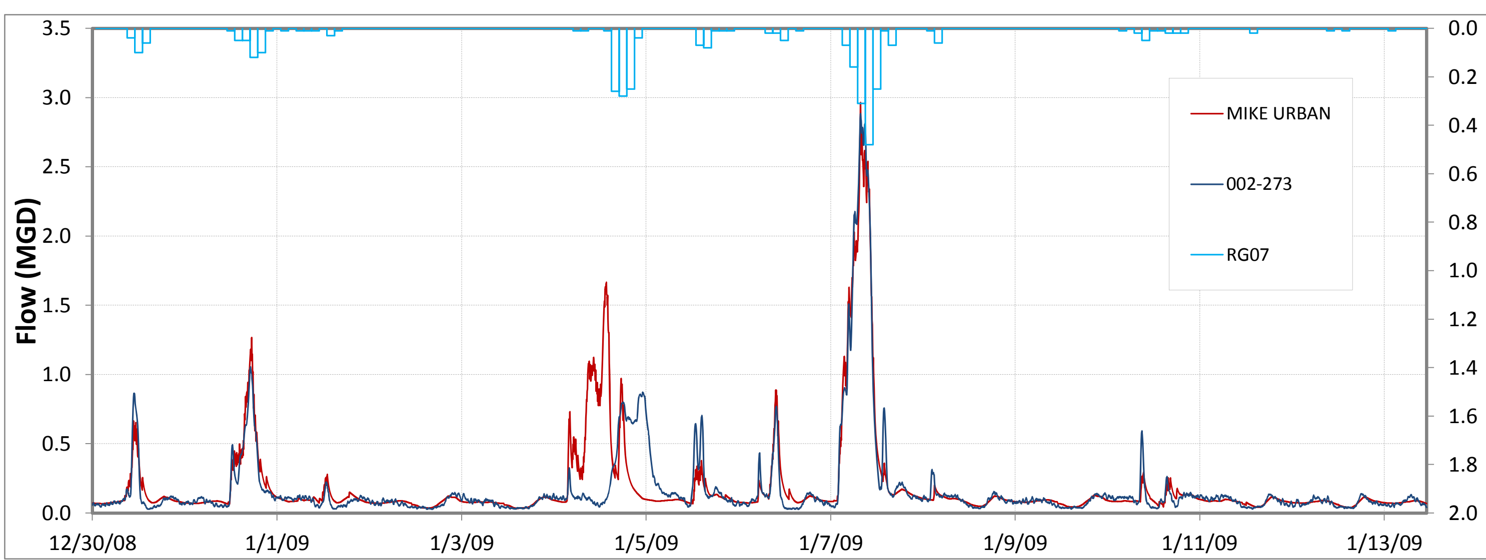


Event 02

10/30/08 to 11/13/08

Parameters	
rdii_area	7.9 %
b_a_iflat	23.0 %
b_m_iflat	0.069
umax	4.66 in
lmax	5.91 in
cqof	1.00
ck	2.7 hr
ckif	1188.1 hr
ckbf	100.0 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.13
RMSE	0.13
Nash	0.75
Q_{pk} Error	51.43%
V_{tot} Error	26.99%

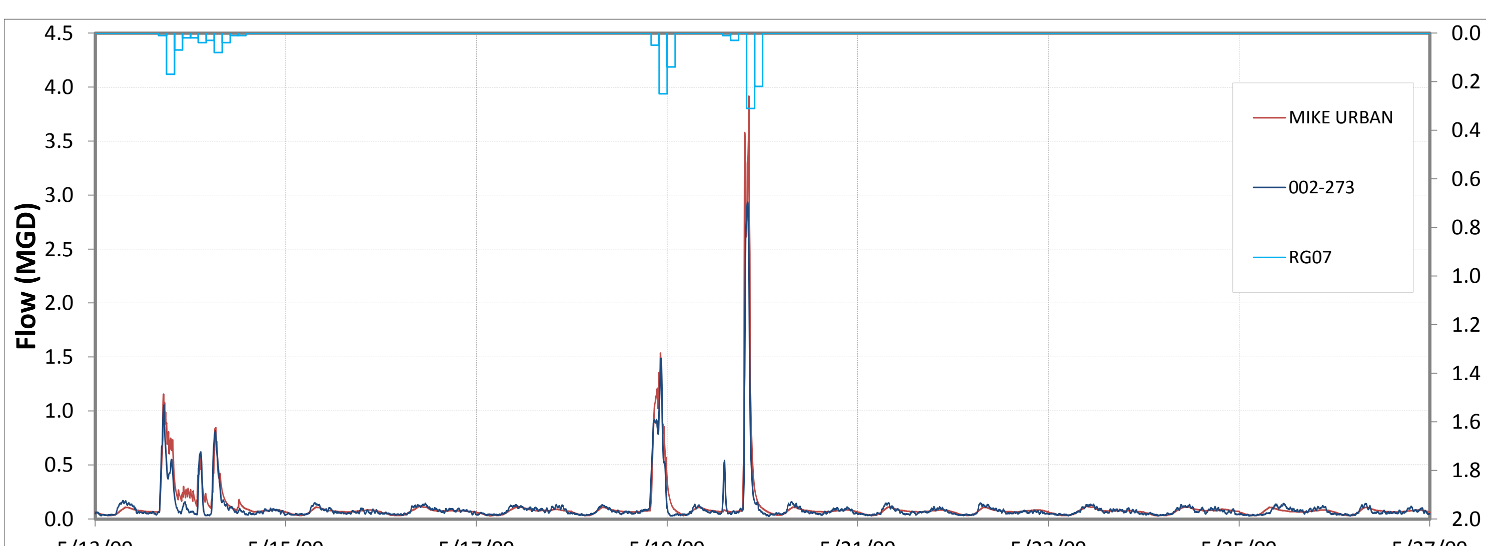


Event 03

12/30/08 to 01/14/09

Parameters	
rdii_area	7.9 %
b_a_iflat	23.0 %
b_m_iflat	0.069
umax	4.66 in
lmax	5.91 in
cqof	1.00
ck	2.7 hr
ckif	1188.1 hr
ckbf	100.0 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.06
RMSE	0.17
Nash	0.71
Q_{pk} Error	2.80%
V_{tot} Error	0.45%

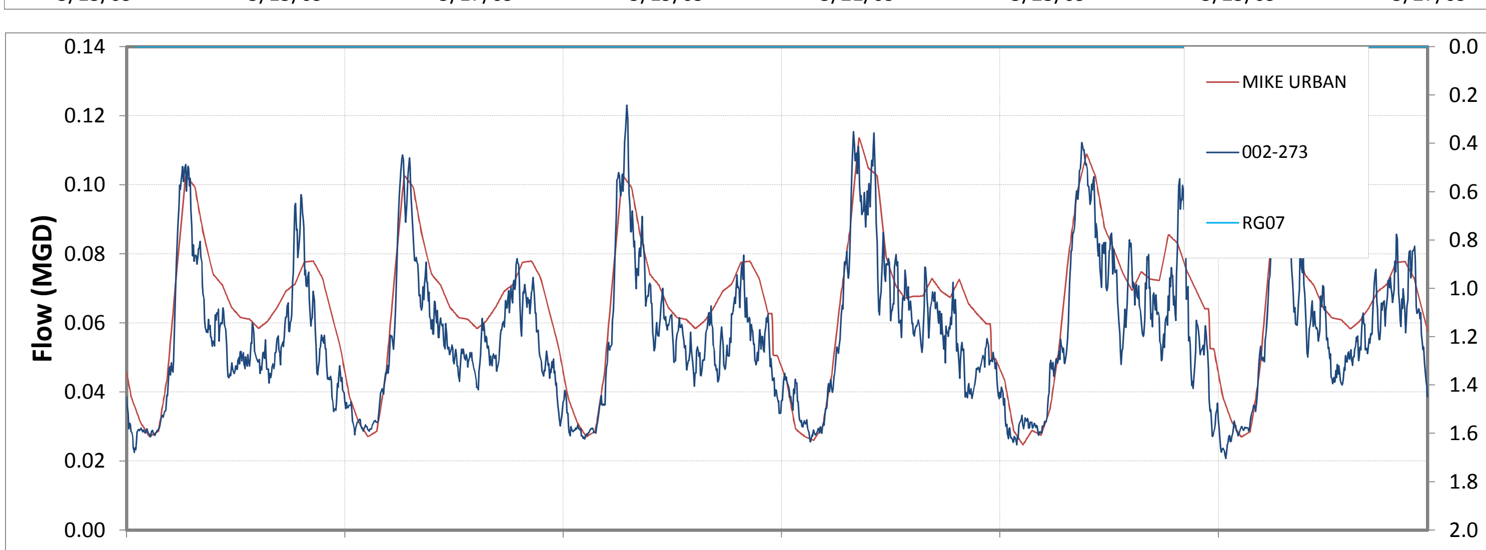


Event 04

05/13/09 to 05/27/09

Parameters	
rdii_area	7.9 %
b_a_iflat	23.0 %
b_m_iflat	0.069
umax	4.66 in
lmax	5.91 in
cqof	1.00
ck	2.7 hr
ckif	1188.1 hr
ckbf	100.0 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.10
RMSE	0.09
Nash	0.81
Q_{pk} Error	33.43%
V_{tot} Error	22.66%



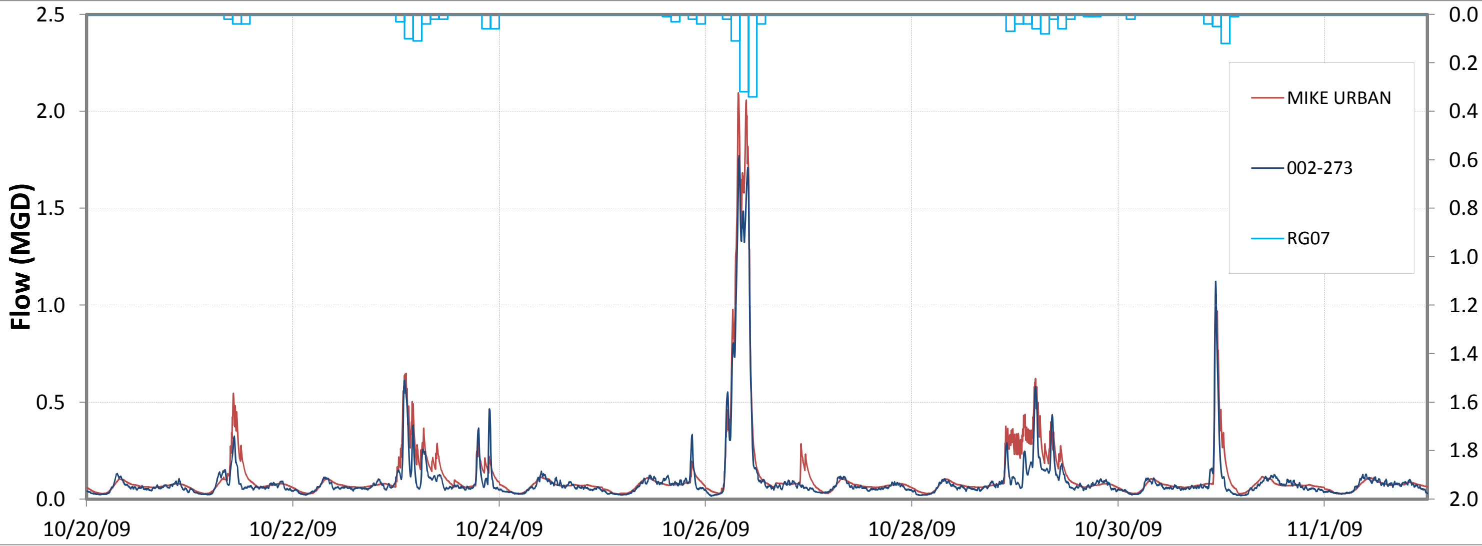
Event 05

09/09/09 to 09/15/09

Parameters	
rdii_area	7.9 %
b_a_iflat	23.0 %
b_m_iflat	0.069
umax	4.66 in
lmax	5.91 in
cqof	1.00
ck	2.7 hr
ckif	1188.1 hr
ckbf	100.0 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.13
RMSE	0.01
Nash	0.68
Q_{pk} Error	-7.68%
V_{tot} Error	12.27%

SB003a

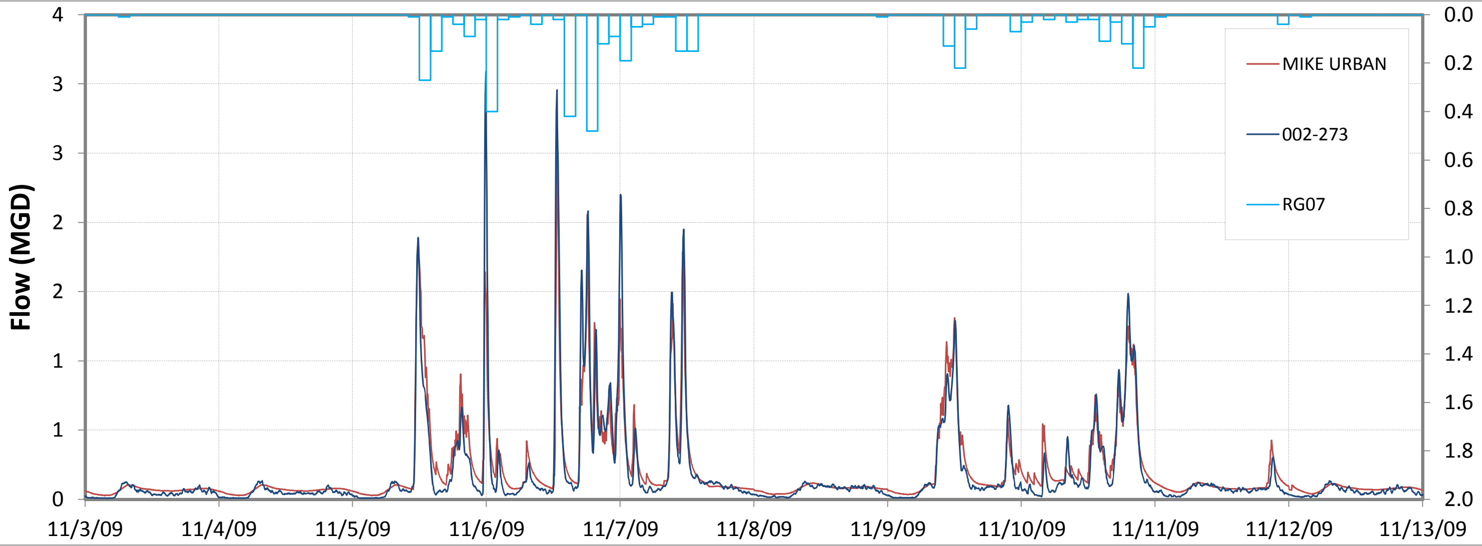


Event 06

10/20/09 to 11/02/09

Parameters	
rdii_area	7.9 %
b_a_iflat	23.0 %
b_m_iflat	0.069
umax	4.66 in
lmax	5.91 in
cqof	1.00
ck	2.7 hr
ckif	1188.1 hr
ckbf	100.0 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.21
RMSE	0.06
Nash	0.88
Q_{pk} Error	18.24%
V_{tot} Error	14.33%

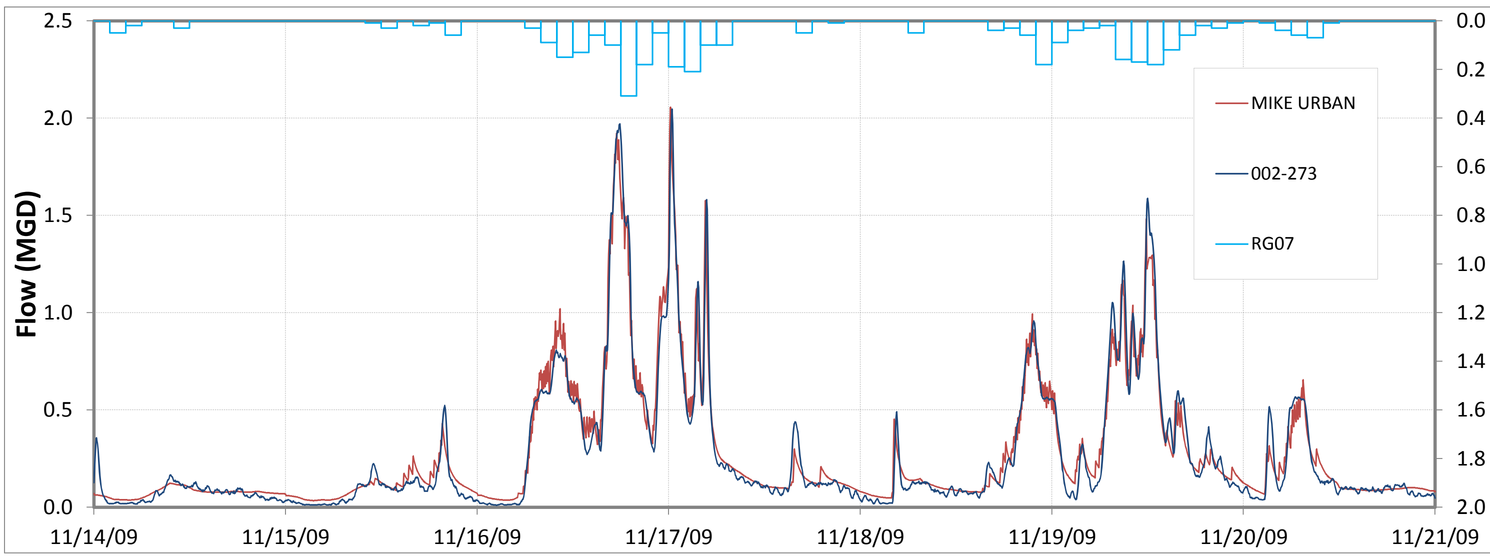


Event 07

11/03/09 to 11/13/09

Parameters	
rdii_area	7.9 %
b_a_iflat	23.0 %
b_m_iflat	0.069
umax	4.66 in
lmax	5.91 in
cqof	1.00
ck	2.7 hr
ckif	1188.1 hr
ckbf	100.0 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.11
RMSE	0.11
Nash	0.88
Q_{pk} Error	-18.72%
V_{tot} Error	0.39%

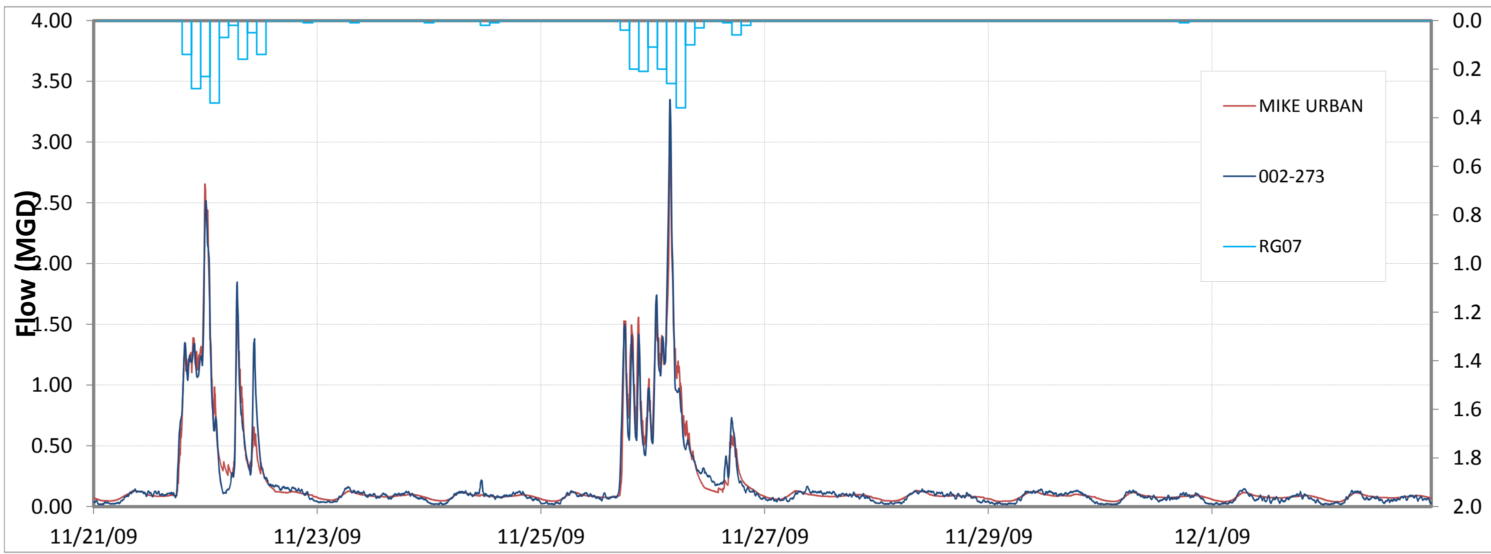


Event 08

11/14/09 to 11/21/09

Parameters	
rdii_area	7.9 %
b_a_iflat	23.0 %
b_m_iflat	0.069
umax	4.66 in
lmax	5.91 in
cqof	1.00
ck	2.7 hr
ckif	1188.1 hr
ckbf	100.0 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.04
RMSE	0.07
Nash	0.96
Q_{pk} Error	0.53%
V_{tot} Error	4.60%

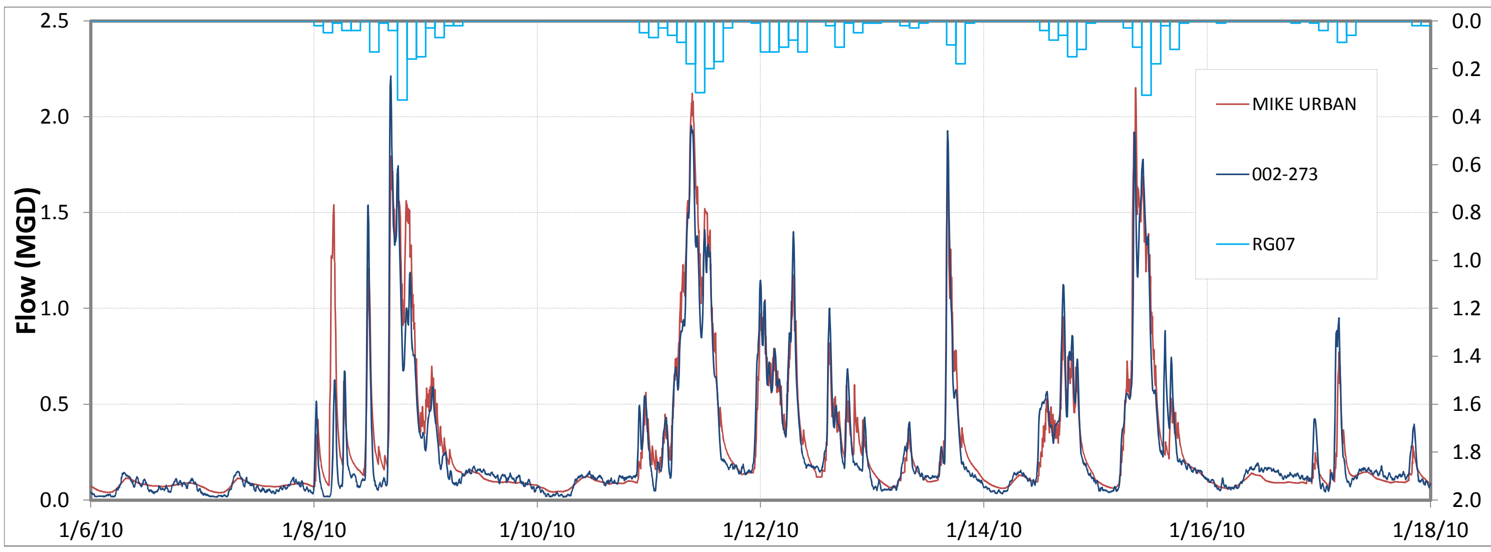


Event 09

11/21/09 to 12/03/09

Parameters	
rdii_area	7.9 %
b_a_iflat	23.0 %
b_m_iflat	0.069
umax	4.66 in
lmax	5.91 in
cqof	1.00
ck	2.7 hr
ckif	1188.1 hr
ckbf	100.0 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.02
RMSE	0.07
Nash	0.96
Q_{pk} Error	-13.60%
V_{tot} Error	0.40%



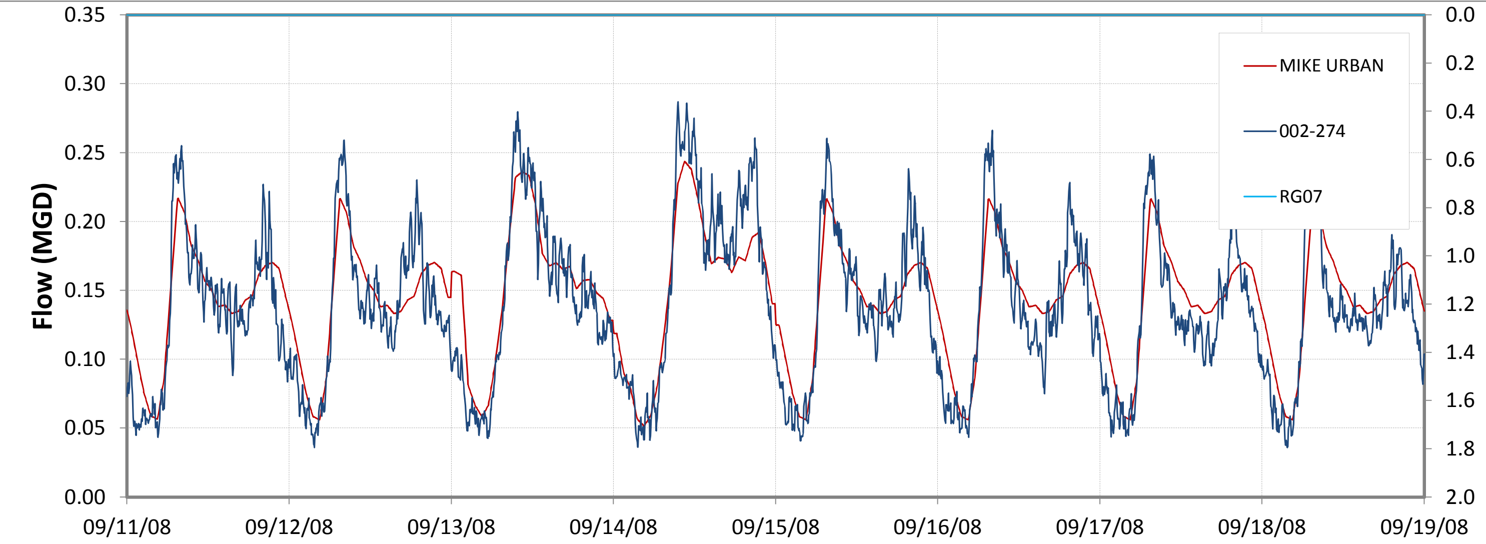
Event 10

01/06/10 to 01/18/10

Parameters	
rdii_area	7.9 %
b_a_iflat	23.0 %
b_m_iflat	0.069
umax	4.66 in
lmax	5.91 in
cqof	1.00
ck	2.7 hr
ckif	1188.1 hr
ckbf	100.0 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.06
RMSE	0.12
Nash	0.88
Q_{pk} Error	-2.79%
V_{tot} Error	-1.11%

Sub-basin: **SB003b** Flow Meter: **002-274**

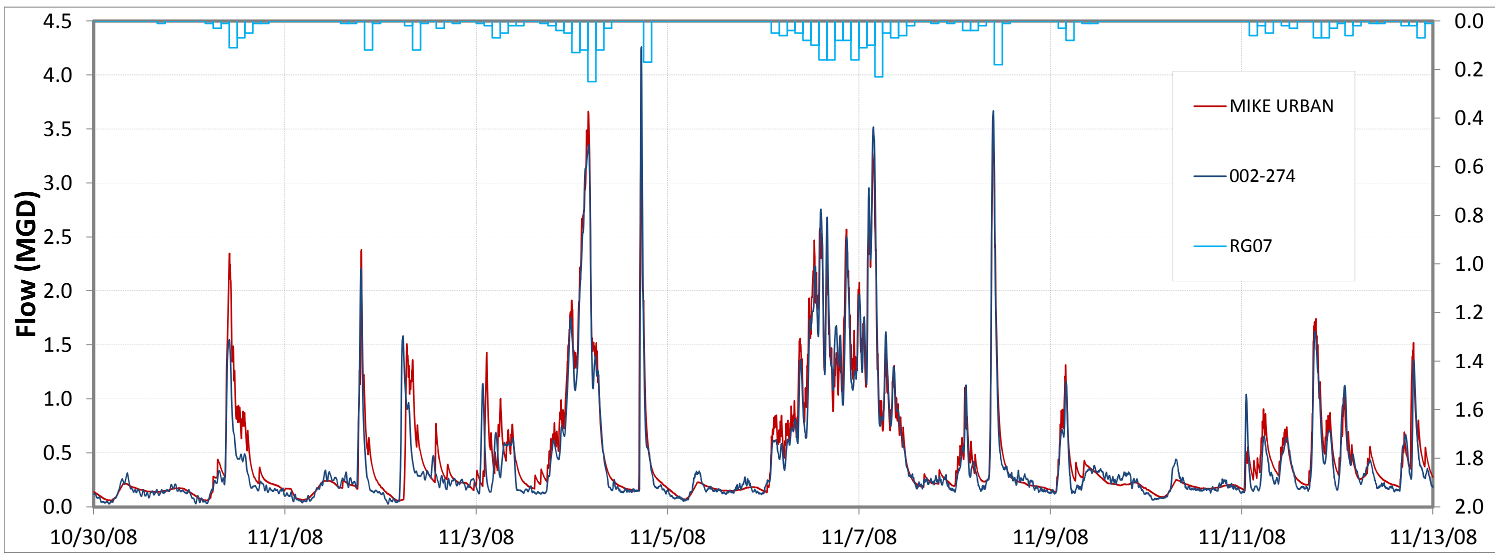


Event 01

09/11/08 to 09/19/08

Parameters	
rdii_area	27.5 %
b_a_iflat	32.0 %
b_m_iflat	0.072
umax	3.92 in
lmax	5.18 in
cqof	0.48
ck	3.1 hr
ckif	341.9 hr
ckbf	846.2 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.04
RMSE	0.03
Nash	0.79
Q_{pk} Error	-15.08%
V_{tot} Error	-1.71%

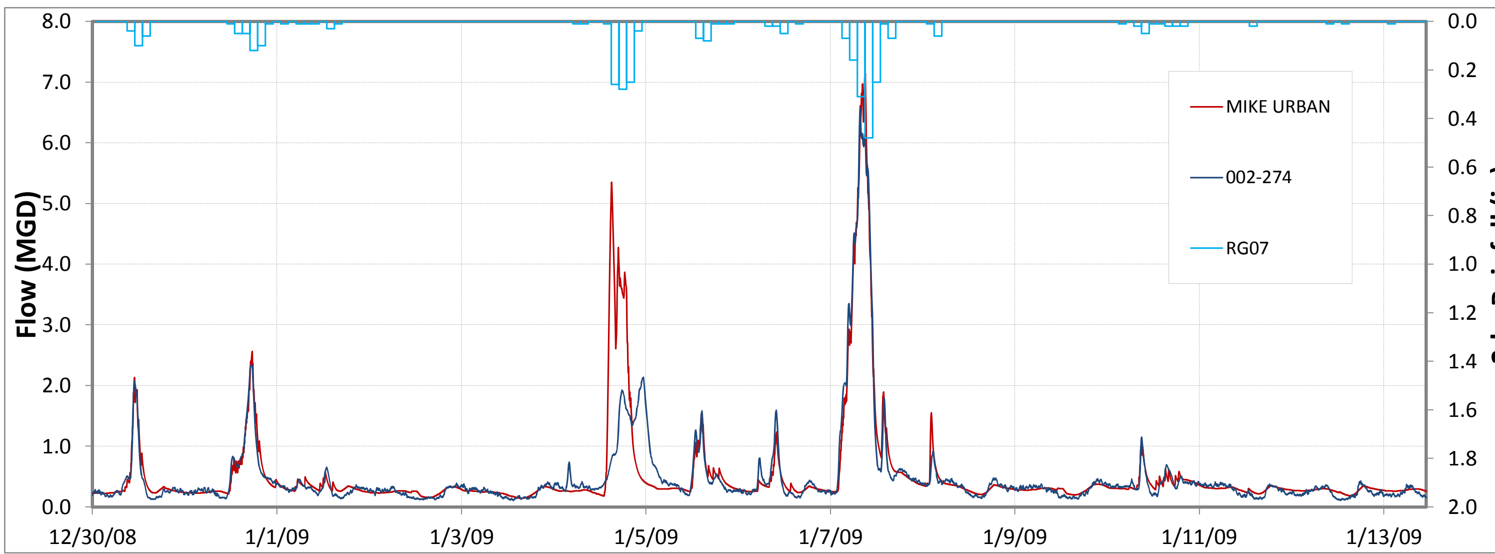


Event 02

10/30/08 to 11/13/08

Parameters	
rdii_area	27.5 %
b_a_iflat	32.0 %
b_m_iflat	0.072
umax	3.92 in
lmax	5.18 in
cqof	0.48
ck	3.1 hr
ckif	341.9 hr
ckbf	846.2 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.10
RMSE	0.17
Nash	0.90
Q_{pk} Error	-14.06%
V_{tot} Error	12.16%

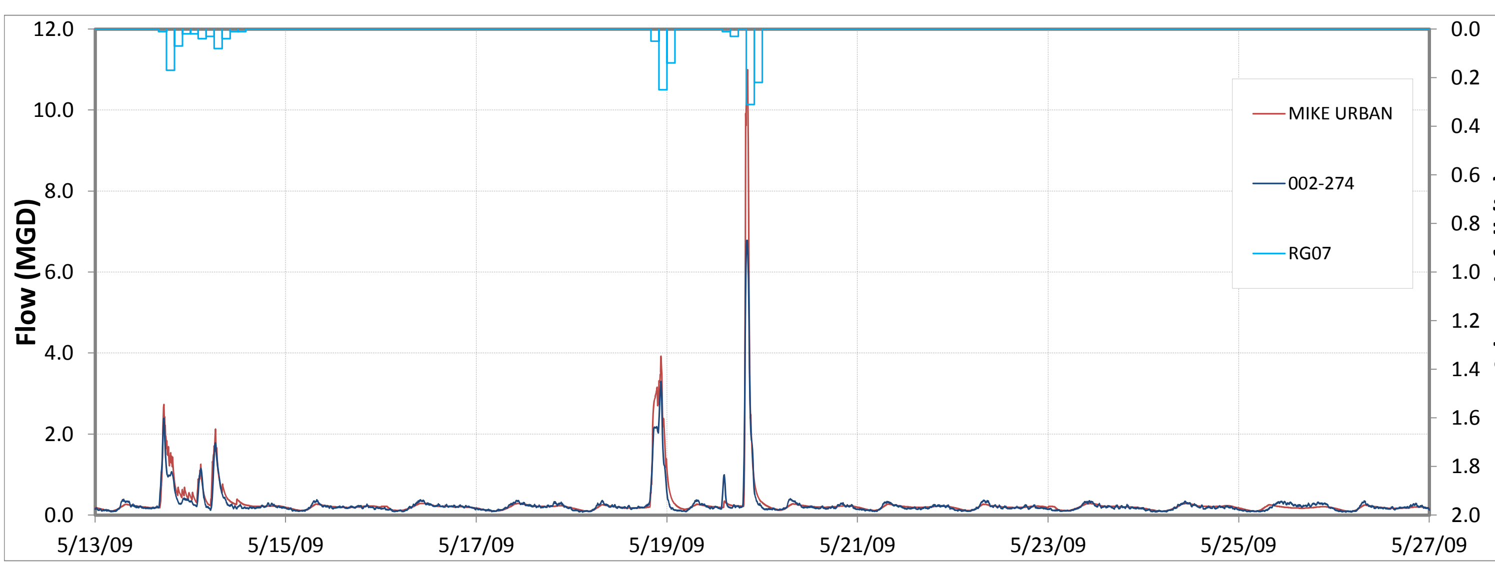


Event 03

12/30/08 to 01/14/09

Parameters	
rdii_area	27.5 %
b_a_iflat	32.0 %
b_m_iflat	0.072
umax	3.92 in
lmax	5.18 in
cqof	0.48
ck	3.1 hr
ckif	341.9 hr
ckbf	846.2 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.06
RMSE	0.37
Nash	0.73
Q_{pk} Error	8.57%
V_{tot} Error	1.17%

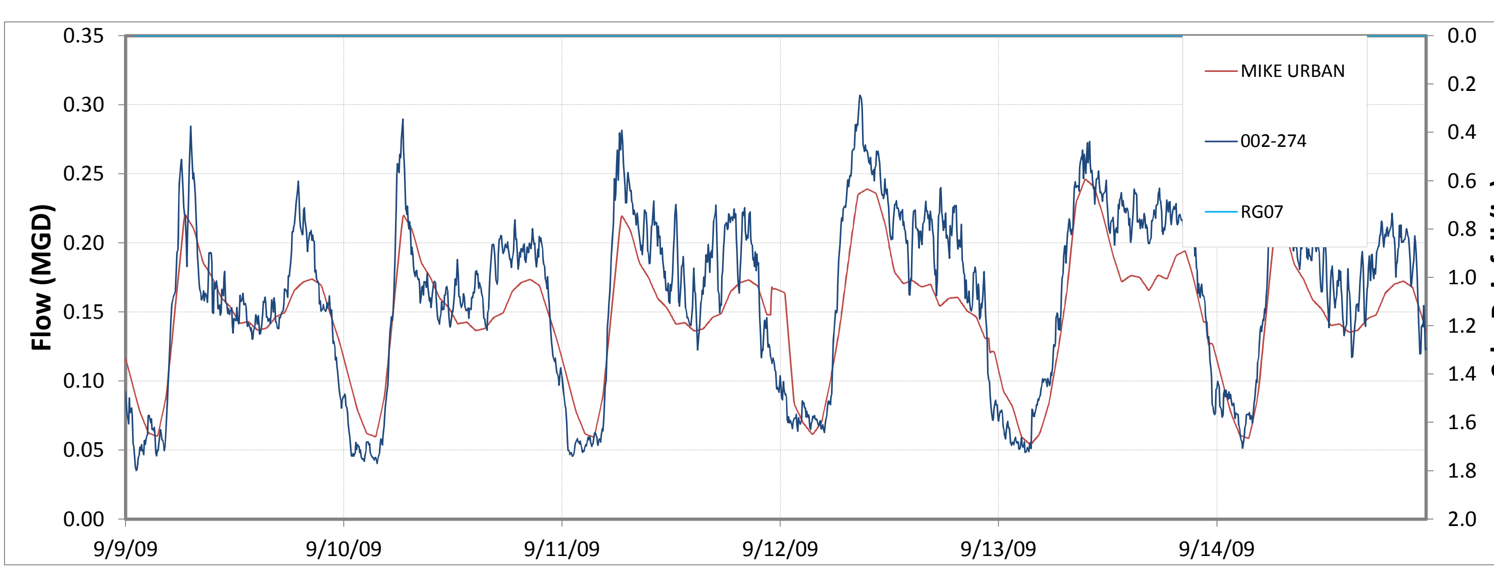


Event 04

05/13/09 to 05/27/09

Parameters	
rdii_area	27.5 %
b_a_iflat	32.0 %
b_m_iflat	0.072
umax	3.92 in
lmax	5.18 in
cqof	0.48
ck	3.1 hr
ckif	341.9 hr
ckbf	846.2 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.10
RMSE	0.22
Nash	0.76
Q_{pk} Error	62.24%
V_{tot} Error	24.74%



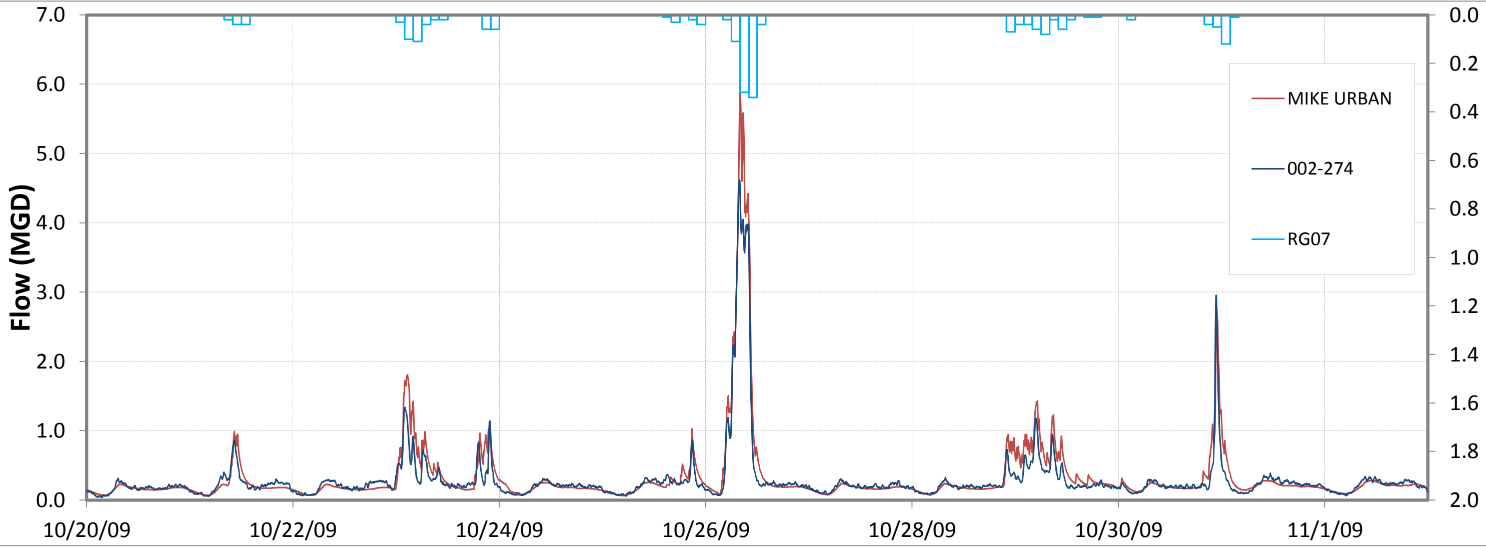
Event 05

09/09/09 to 09/15/09

Parameters	
rdii_area	27.5 %
b_a_iflat	32.0 %
b_m_iflat	0.072
umax	3.92 in
lmax	5.18 in
cqof	0.48
ck	3.1 hr
ckif	341.9 hr
ckbf	846.2 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	-0.09
RMSE	0.03
Nash	0.73
Q_{pk} Error	-20.02%
V_{tot} Error	-12.35%

SB003b

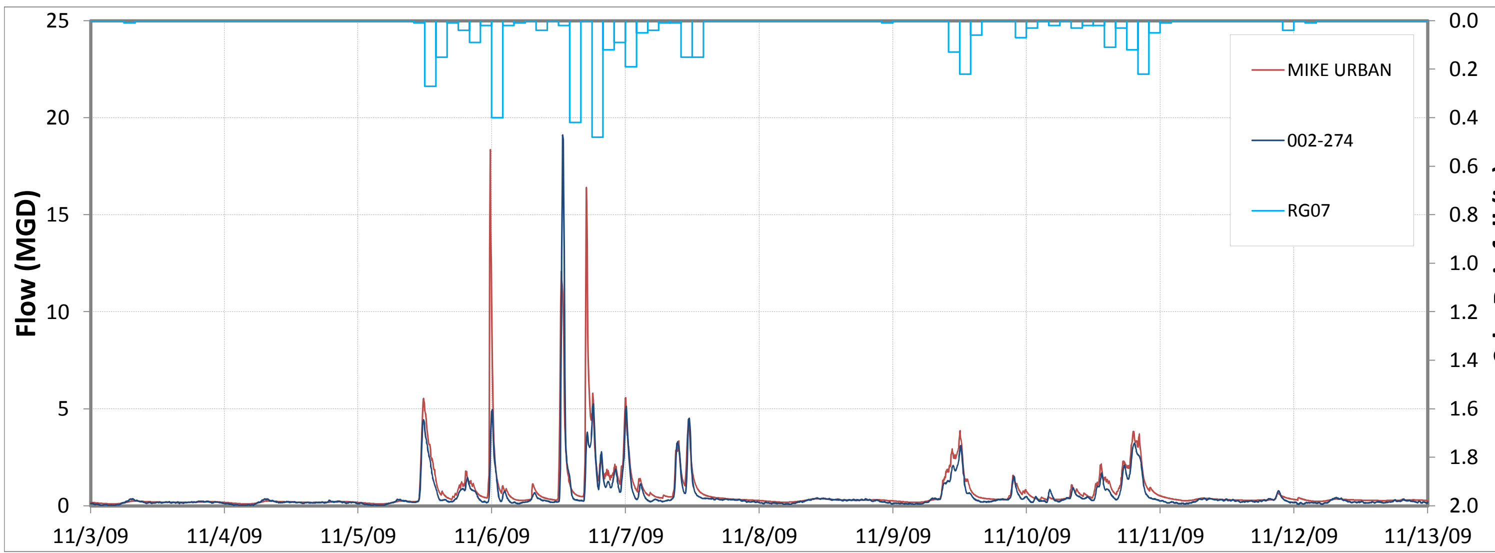


Event 06

10/20/09 to 11/02/09

Parameters	
rdii_area	27.5 %
b_a_iflat	32.0 %
b_m_iflat	0.072
umax	3.92 in
lmax	5.18 in
cqof	0.48
ck	3.1 hr
ckif	341.9 hr
ckbf	846.2 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.11
RMSE	0.16
Nash	0.87
Q_{pk} Error	31.13%
V_{tot} Error	14.66%

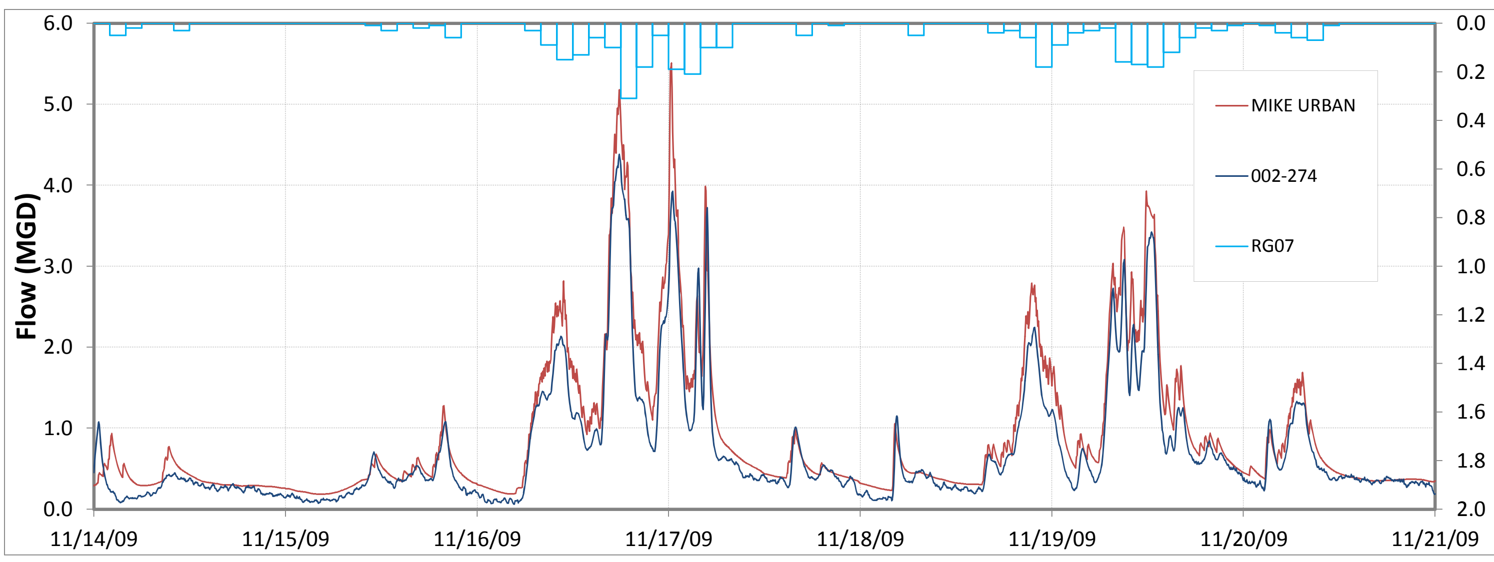


Event 07

11/03/09 to 11/13/09

Parameters	
rdii_area	27.5 %
b_a_iflat	32.0 %
b_m_iflat	0.072
umax	3.92 in
lmax	5.18 in
cqof	0.48
ck	3.1 hr
ckif	341.9 hr
ckbf	846.2 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.31
RMSE	0.78
Nash	0.45
Q_{pk} Error	-3.97%
V_{tot} Error	38.39%

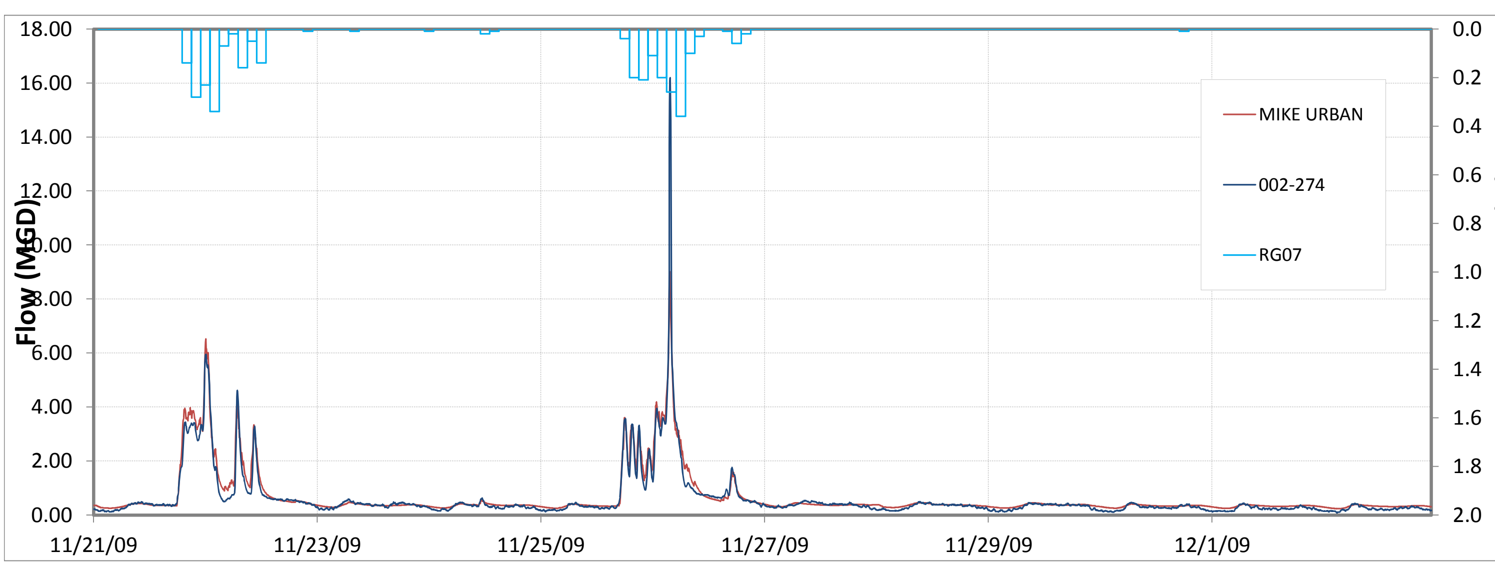


Event 08

11/14/09 to 11/21/09

Parameters	
rdii_area	27.5 %
b_a_iflat	32.0 %
b_m_iflat	0.072
umax	3.92 in
lmax	5.18 in
cqof	0.48
ck	3.1 hr
ckif	341.9 hr
ckbf	846.2 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.25
RMSE	0.28
Nash	0.87
Q_{pk} Error	25.80%
V_{tot} Error	23.09%

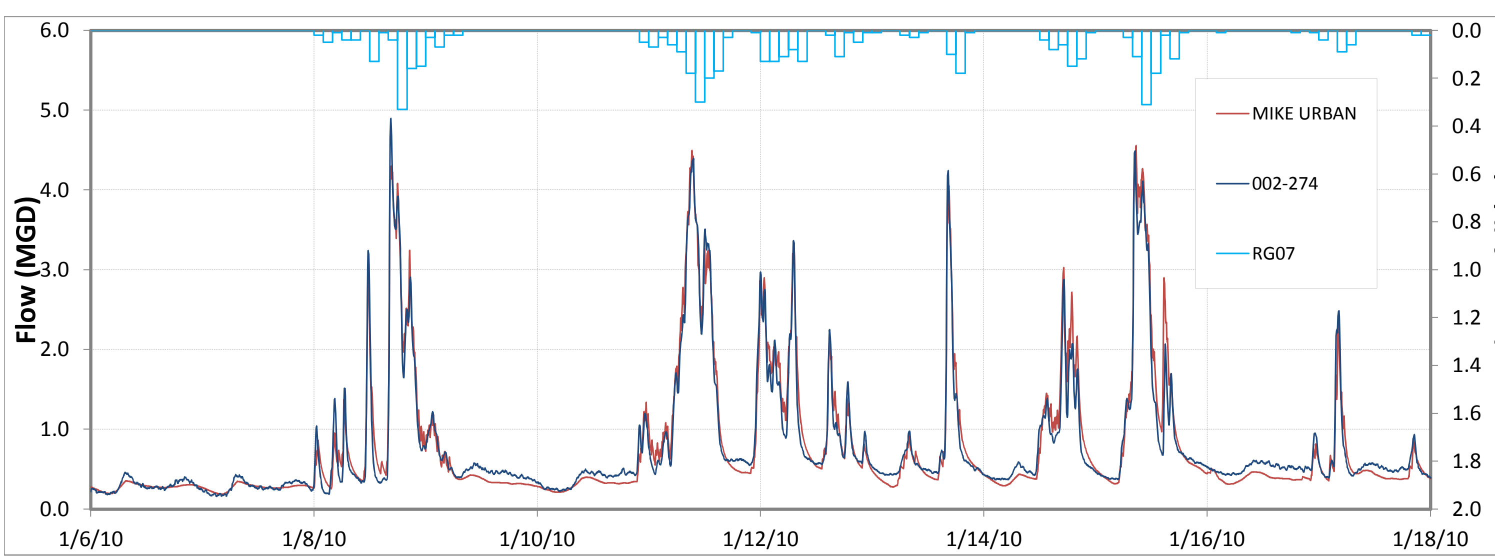


Event 09

11/21/09 to 12/03/09

Parameters	
rdii_area	27.5 %
b_a_iflat	32.0 %
b_m_iflat	0.072
umax	3.92 in
lmax	5.18 in
cqof	0.48
ck	3.1 hr
ckif	341.9 hr
ckbf	846.2 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.10
RMSE	0.30
Nash	0.91
Q_{pk} Error	-44.32%
V_{tot} Error	1.04%



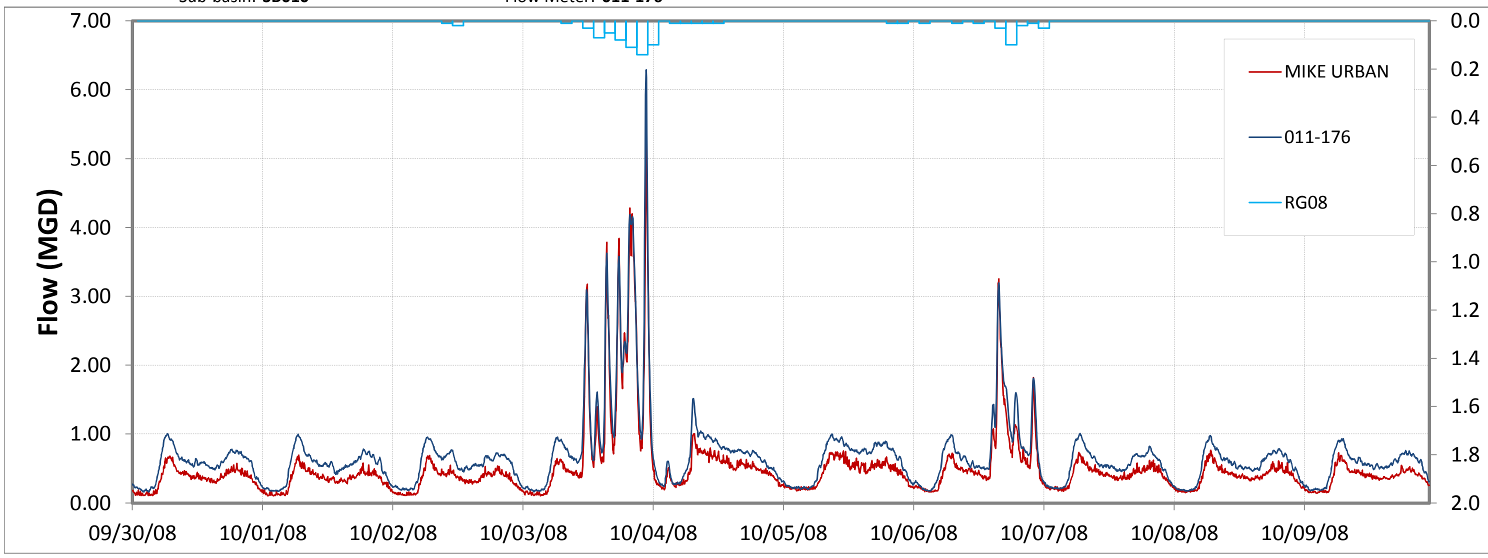
Event 10

01/06/10 to 01/18/10

Parameters	
rdii_area	27.5 %
b_a_iflat	32.0 %
b_m_iflat	0.072
umax	3.92 in
lmax	5.18 in
cqof	0.48
ck	3.1 hr
ckif	341.9 hr
ckbf	846.2 hr
i_u	0.39 in
i_l	5.91 in
gw_carea	1.000

Goodness-of-Fit	
Bias	-0.01
RMSE	0.16
Nash	0.96
Q_{pk} Error	-6.94%
V_{tot} Error	7.95%

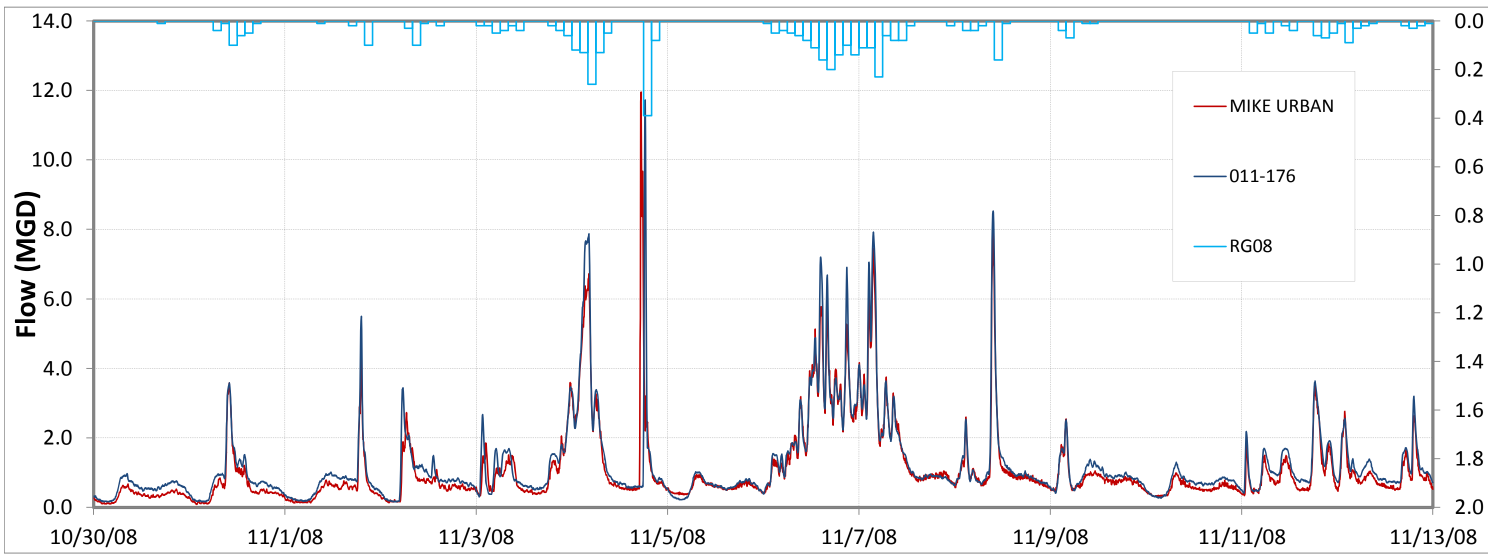
Sub-basin: **SB010** Flow Meter: **011-176**



Event 01
09/30/08 to 10/10/08

Parameters	
rdii_area	114.8 %
b_a_iflat	18.0 %
b_m_iflat	0.040
umax	0.20 in
lmax	13.84 in
cqof	0.22
ck	17.2 hr
ckif	146.7 hr
ckbf	1700.7 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

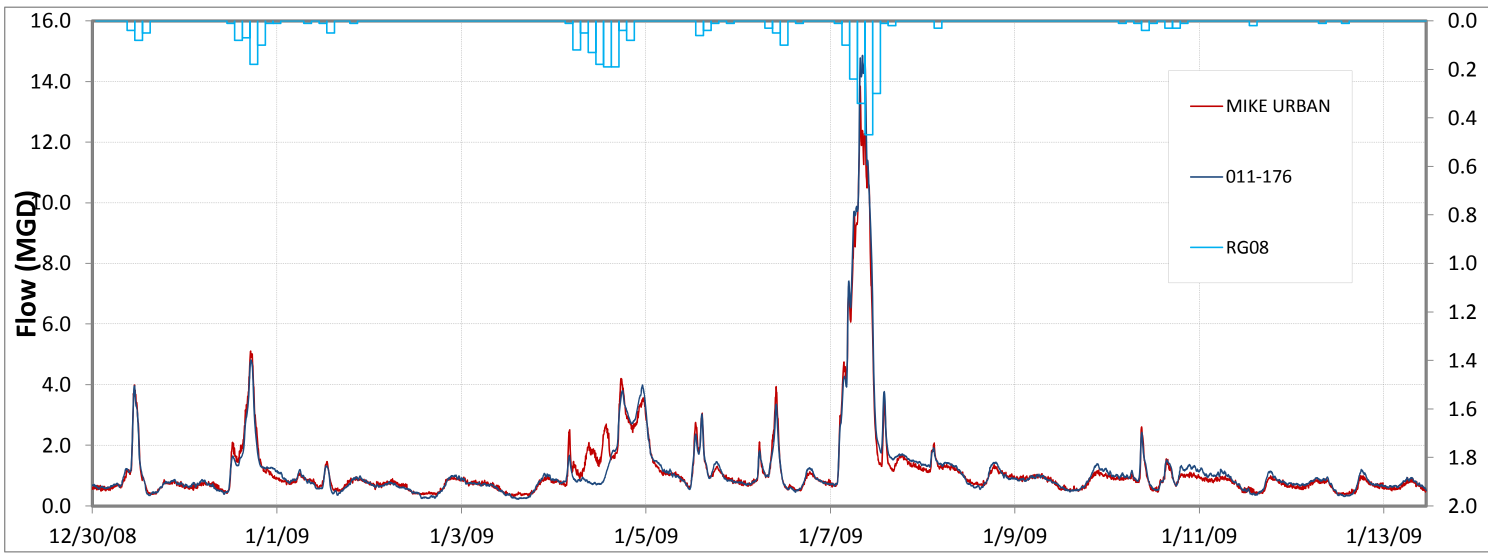
Goodness-of-Fit	
Bias	-0.26
RMSE	0.20
Nash	0.86
Q_{pk} Error	-10.11%
V_{tot} Error	-17.32%



Event 02
10/30/08 to 11/13/08

Parameters	
rdii_area	114.8 %
b_a_iflat	18.0 %
b_m_iflat	0.040
umax	0.20 in
lmax	13.84 in
cqof	0.22
ck	17.2 hr
ckif	146.7 hr
ckbf	1700.7 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

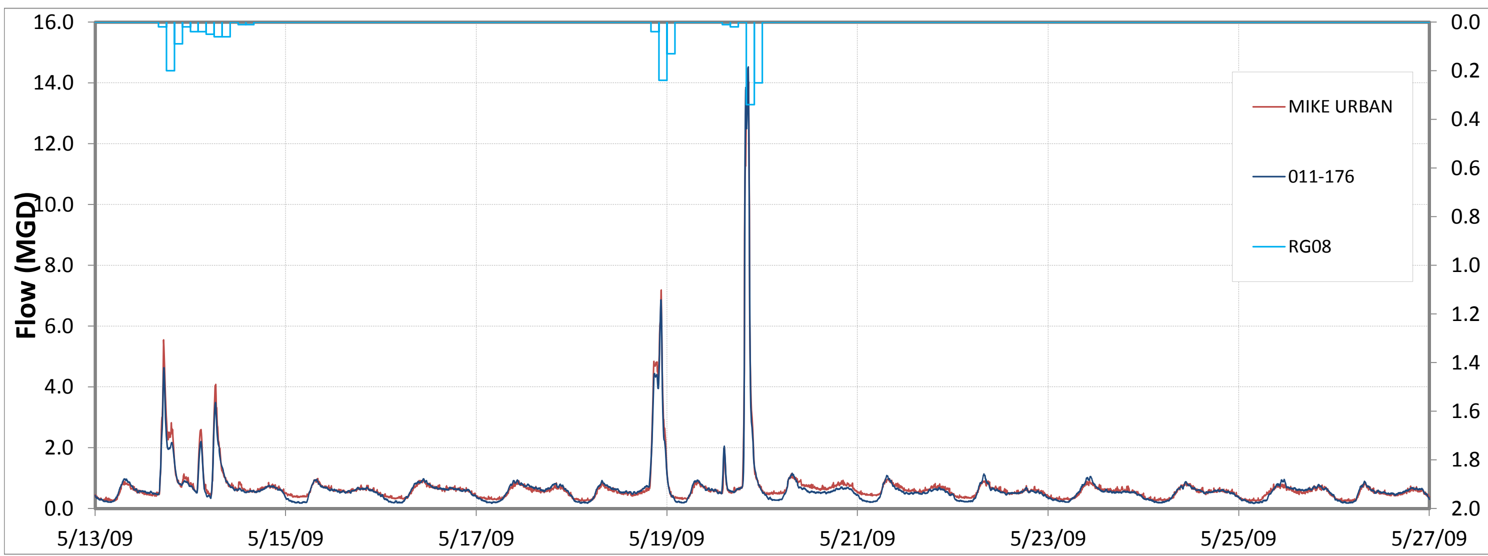
Goodness-of-Fit	
Bias	-0.11
RMSE	0.59
Nash	0.76
Q_{pk} Error	1.95%
V_{tot} Error	-1.90%



Event 03
12/30/08 to 01/14/09

Parameters	
rdii_area	114.8 %
b_a_iflat	18.0 %
b_m_iflat	0.040
umax	0.20 in
lmax	13.84 in
cqof	0.22
ck	17.2 hr
ckif	146.7 hr
ckbf	1700.7 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

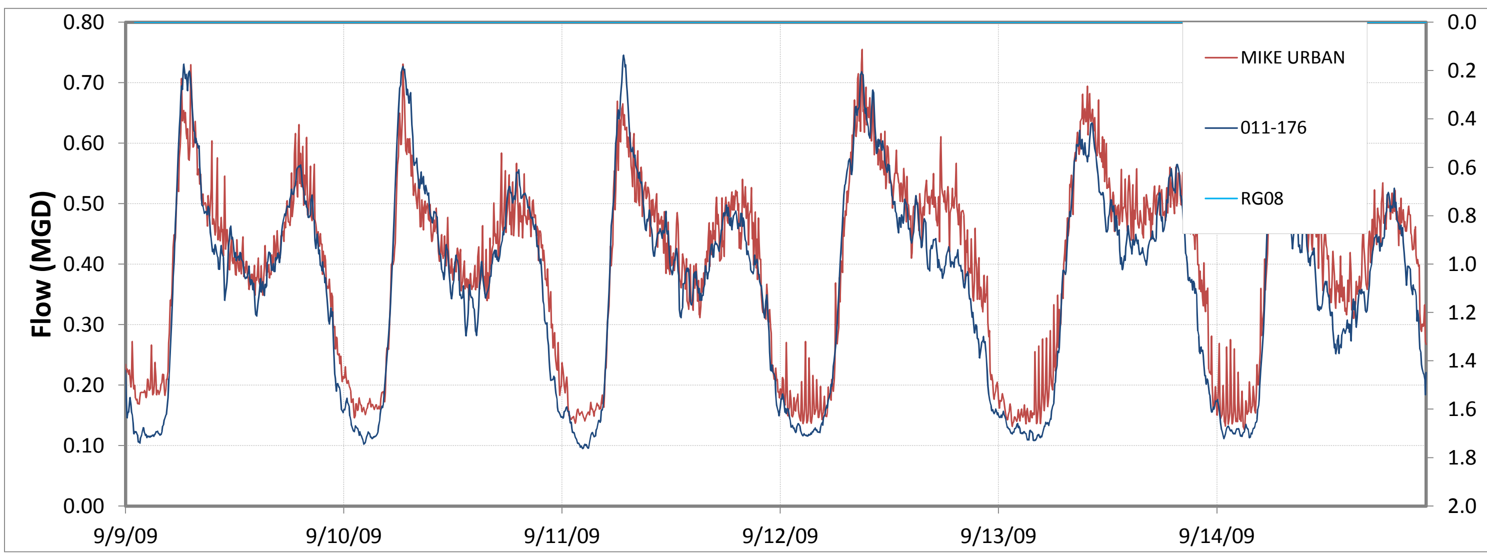
Goodness-of-Fit	
Bias	-0.02
RMSE	0.26
Nash	0.97
Q_{pk} Error	-6.84%
V_{tot} Error	-6.60%



Event 04
05/13/09 to 05/27/09

Parameters	
rdii_area	114.8 %
b_a_iflat	18.0 %
b_m_iflat	0.040
umax	0.20 in
lmax	13.84 in
cqof	0.22
ck	17.2 hr
ckif	146.7 hr
ckbf	1700.7 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.05
RMSE	0.14
Nash	0.98
Q_{pk} Error	-0.34%
V_{tot} Error	7.12%

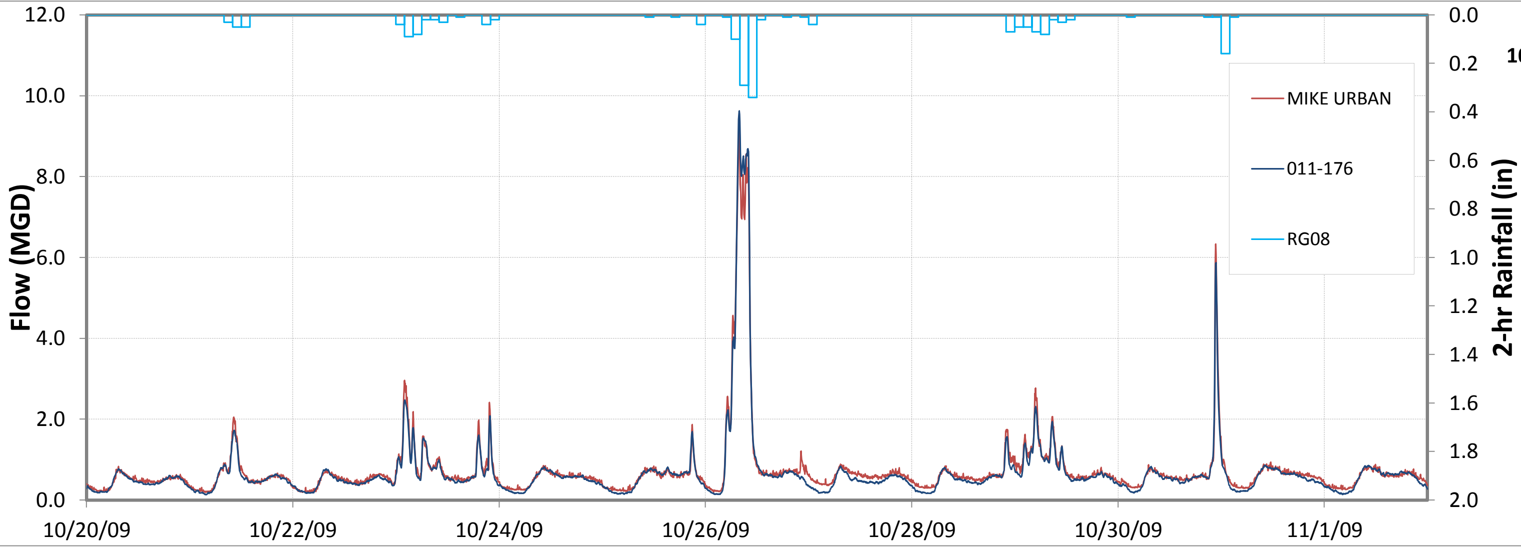


Event 05
09/09/09 to 09/15/09

Parameters	
rdii_area	114.8 %
b_a_iflat	18.0 %
b_m_iflat	0.040
umax	0.20 in
lmax	13.84 in
cqof	0.22
ck	17.2 hr
ckif	146.7 hr
ckbf	1700.7 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.08
RMSE	0.06
Nash	0.88
Q_{pk} Error	1.28%
V_{tot} Error	8.68%

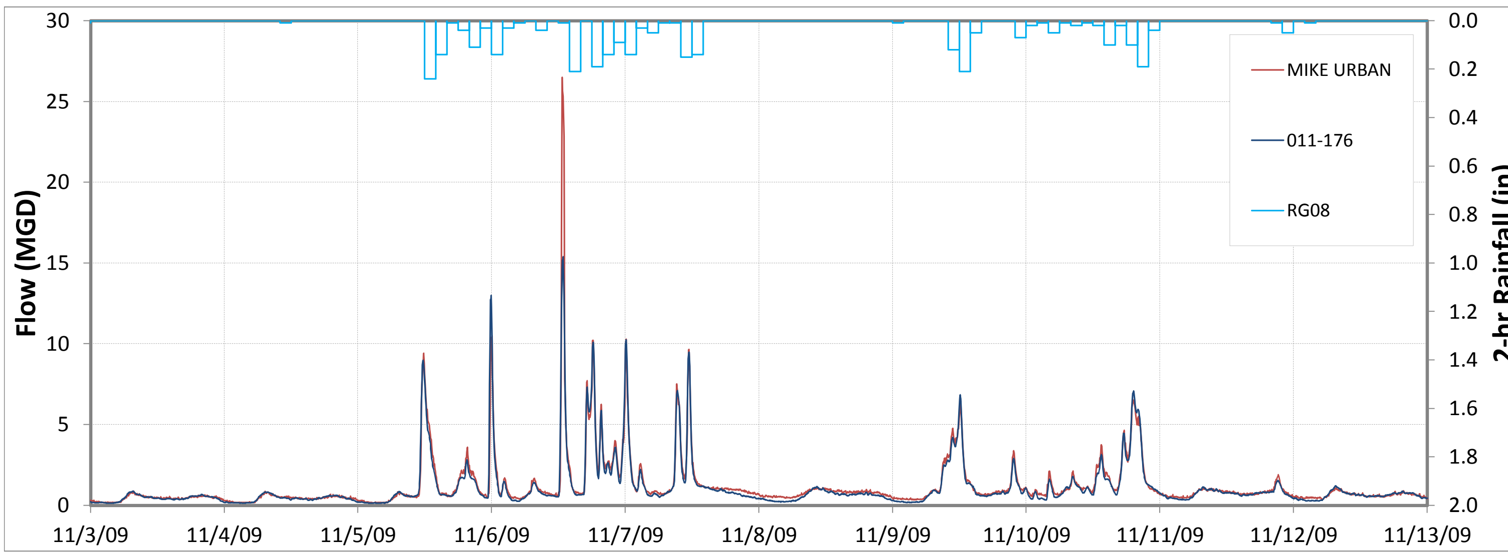
SB010



Event 06
10/20/09 to 11/02/09

Parameters	
rdii_area	114.8 %
b_a_iflat	18.0 %
b_m_iflat	0.040
umax	0.20 in
lmax	13.84 in
cqof	0.22
ck	17.2 hr
ckif	146.7 hr
ckbf	1700.7 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

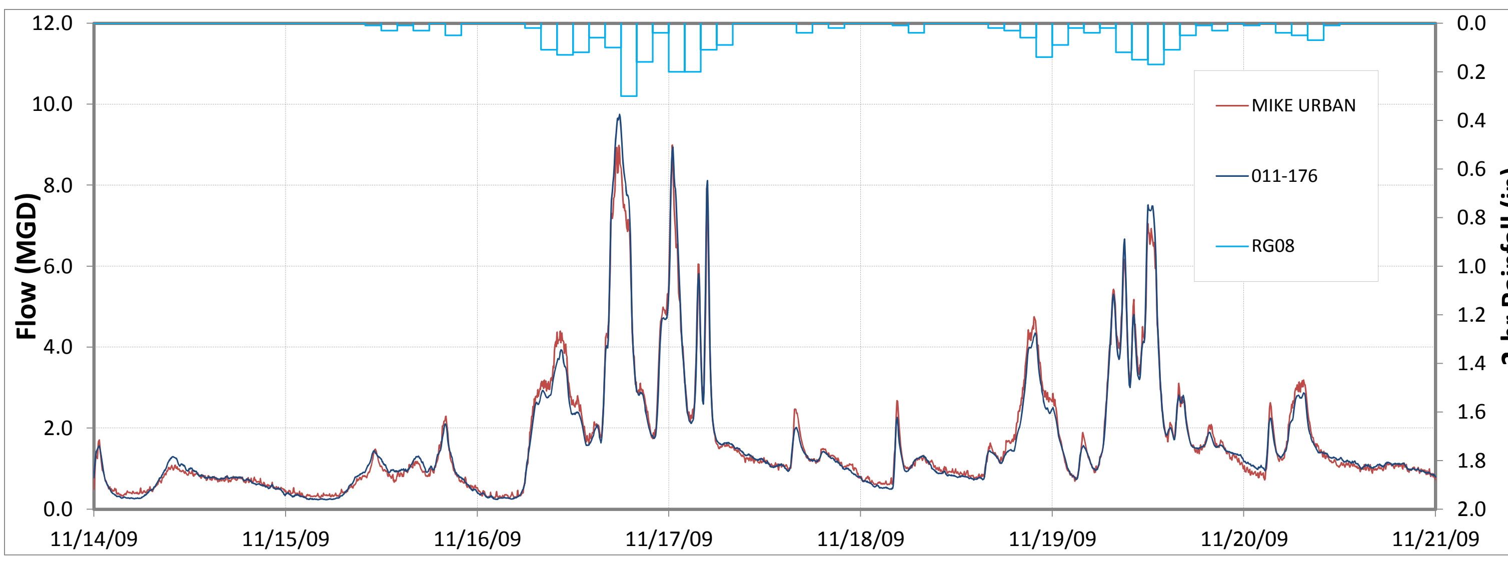
Goodness-of-Fit	
Bias	0.07
RMSE	0.12
Nash	0.98
Q_{pk} Error	-2.27%
V_{tot} Error	2.00%



Event 07
11/03/09 to 11/13/09

Parameters	
rdii_area	114.8 %
b_a_iflat	18.0 %
b_m_iflat	0.040
umax	0.20 in
lmax	13.84 in
cqof	0.22
ck	17.2 hr
ckif	146.7 hr
ckbf	1700.7 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

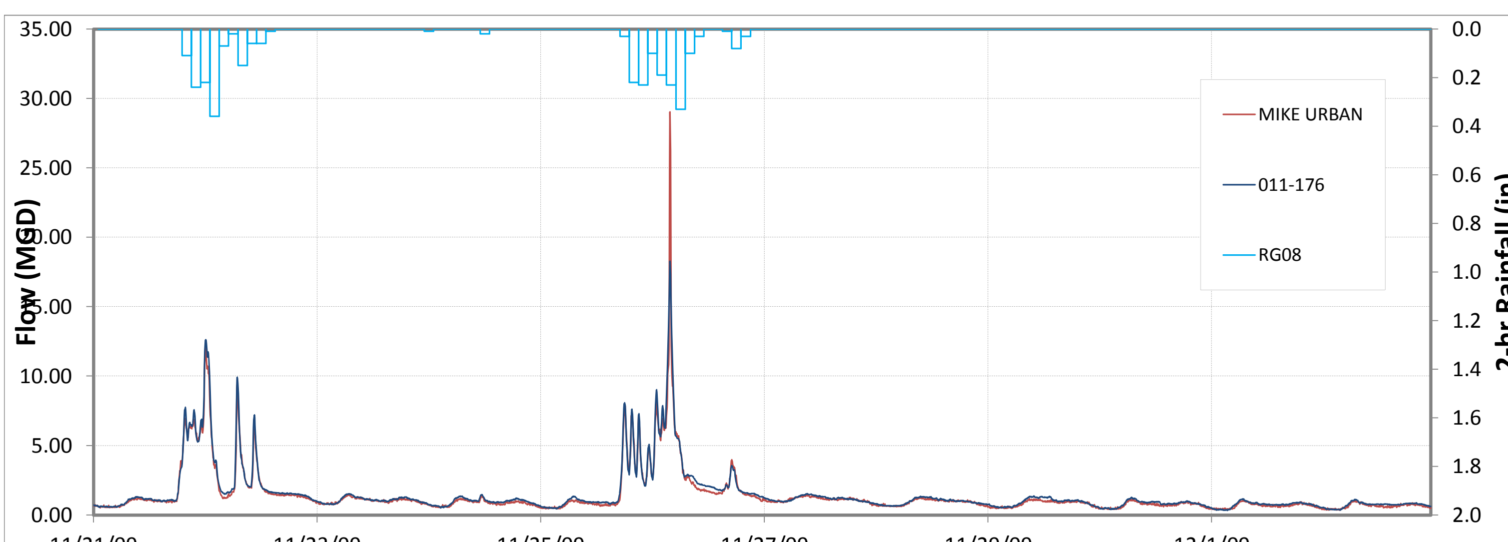
Goodness-of-Fit	
Bias	0.09
RMSE	0.53
Nash	0.89
Q_{pk} Error	72.03%
V_{tot} Error	8.45%



Event 08
11/14/09 to 11/21/09

Parameters	
rdii_area	114.8 %
b_a_iflat	18.0 %
b_m_iflat	0.040
umax	0.20 in
lmax	13.84 in
cqof	0.22
ck	17.2 hr
ckif	146.7 hr
ckbf	1700.7 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

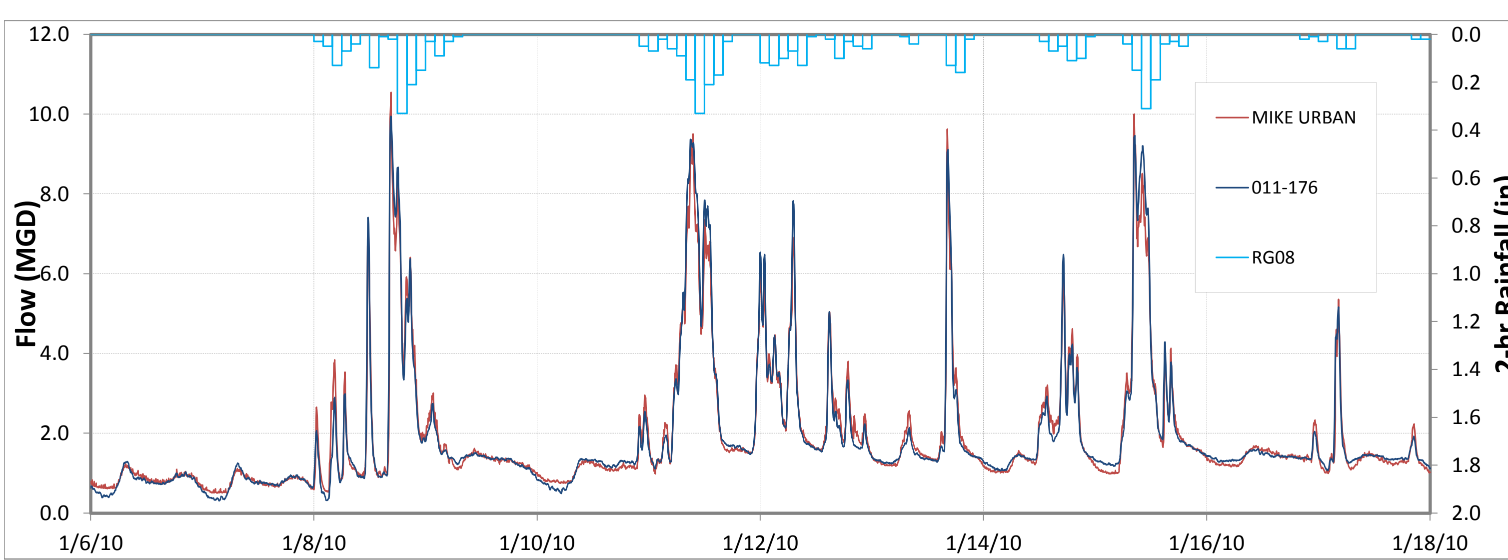
Goodness-of-Fit	
Bias	0.01
RMSE	0.19
Nash	0.98
Q_{pk} Error	-7.77%
V_{tot} Error	0.93%



Event 09
11/21/09 to 12/03/09

Parameters	
rdii_area	114.8 %
b_a_iflat	18.0 %
b_m_iflat	0.040
umax	0.20 in
lmax	13.84 in
cqof	0.22
ck	17.2 hr
ckif	146.7 hr
ckbf	1700.7 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	-0.06
RMSE	0.35
Nash	0.96
Q_{pk} Error	58.85%
V_{tot} Error	-4.82%

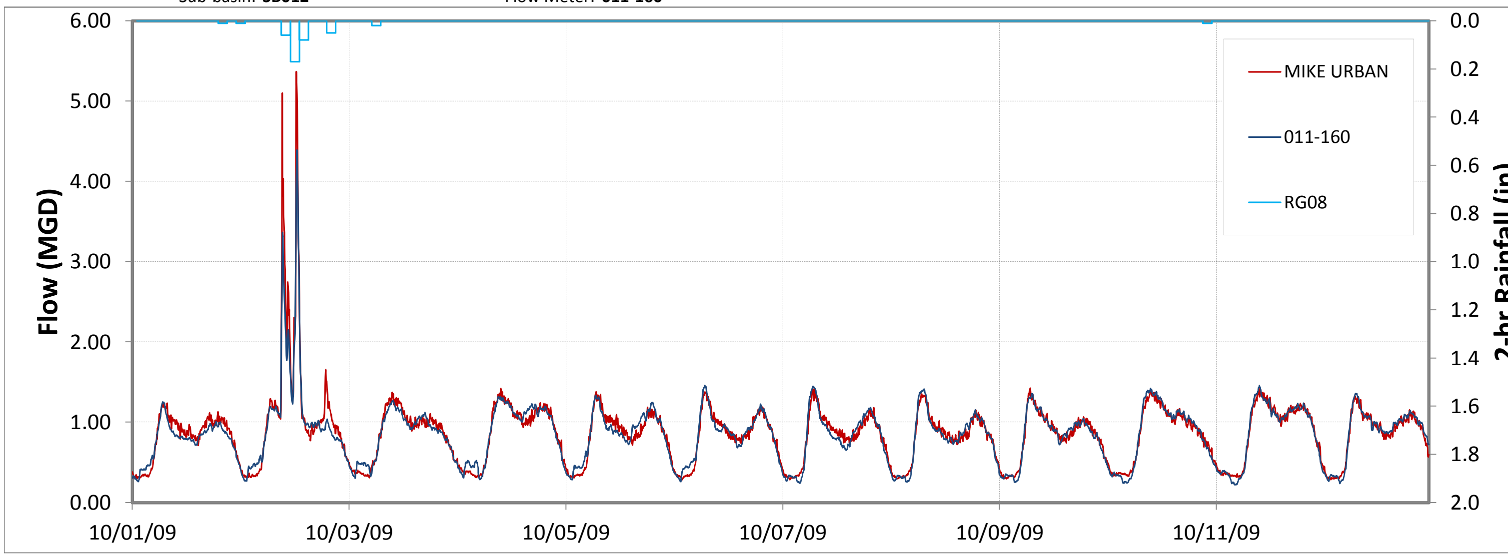


Event 10
01/06/10 to 01/18/10

Parameters	
rdii_area	114.8 %
b_a_iflat	18.0 %
b_m_iflat	0.040
umax	0.20 in
lmax	13.84 in
cqof	0.22
ck	17.2 hr
ckif	146.7 hr
ckbf	1700.7 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.00
RMSE	0.24
Nash	0.98
Q_{pk} Error	6.06%
V_{tot} Error	1.81%

Sub-basin: **SB012** Flow Meter: **011-160**

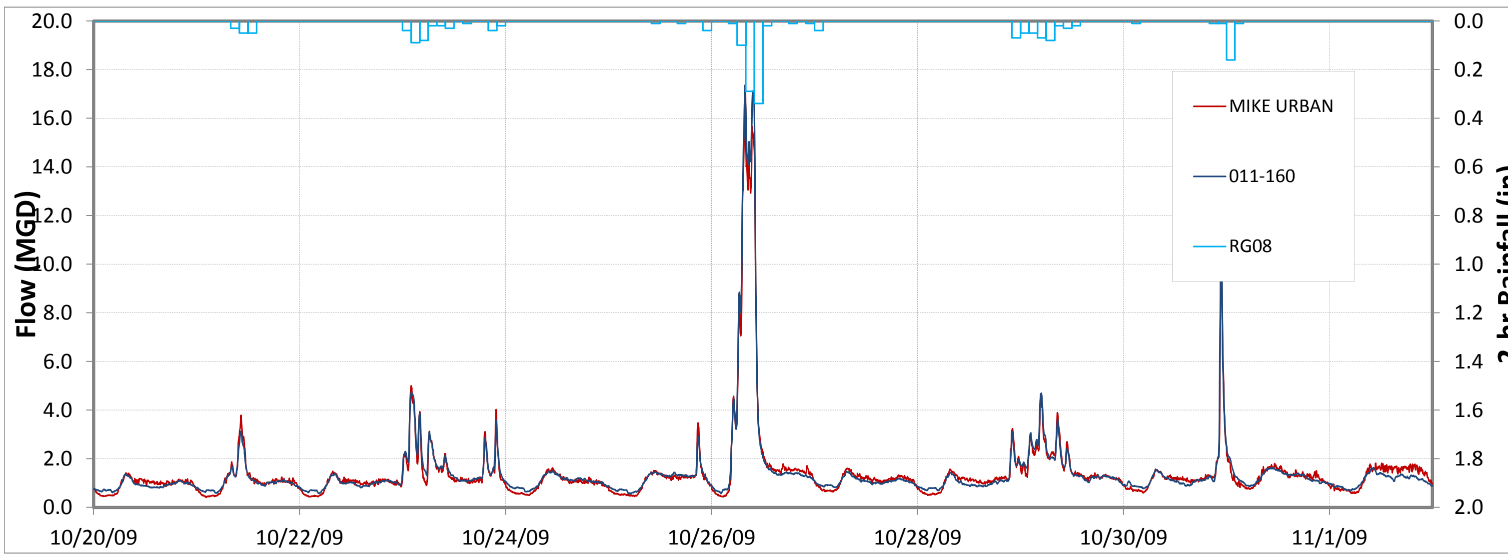


Event 01

10/01/09 to 10/13/09

Parameters	
rdii_area	45.6 %
b_a_iflat	7.6 %
b_m_iflat	0.079
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.01
RMSE	0.10
Nash	0.92
Q_{pk} Error	22.22%
V_{tot} Error	6.56%

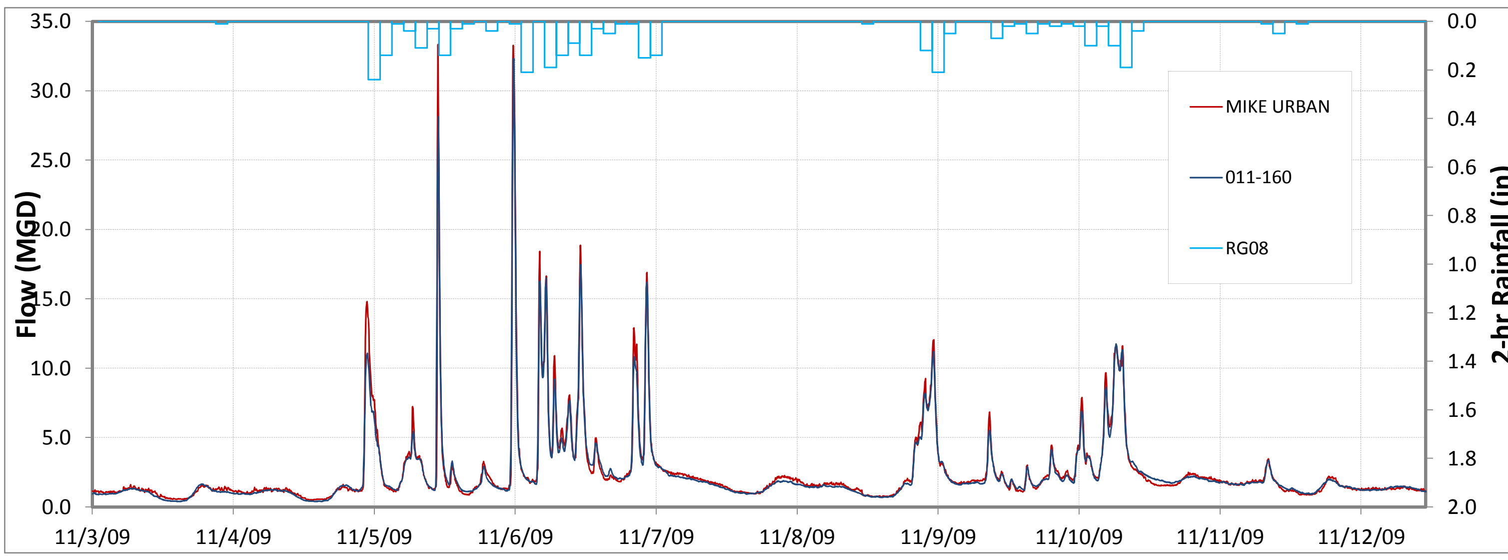


Event 02

10/20/09 to 11/02/09

Parameters	
rdii_area	45.6 %
b_a_iflat	7.6 %
b_m_iflat	0.079
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.00
RMSE	0.20
Nash	0.99
Q_{pk} Error	-0.84%
V_{tot} Error	-2.54%

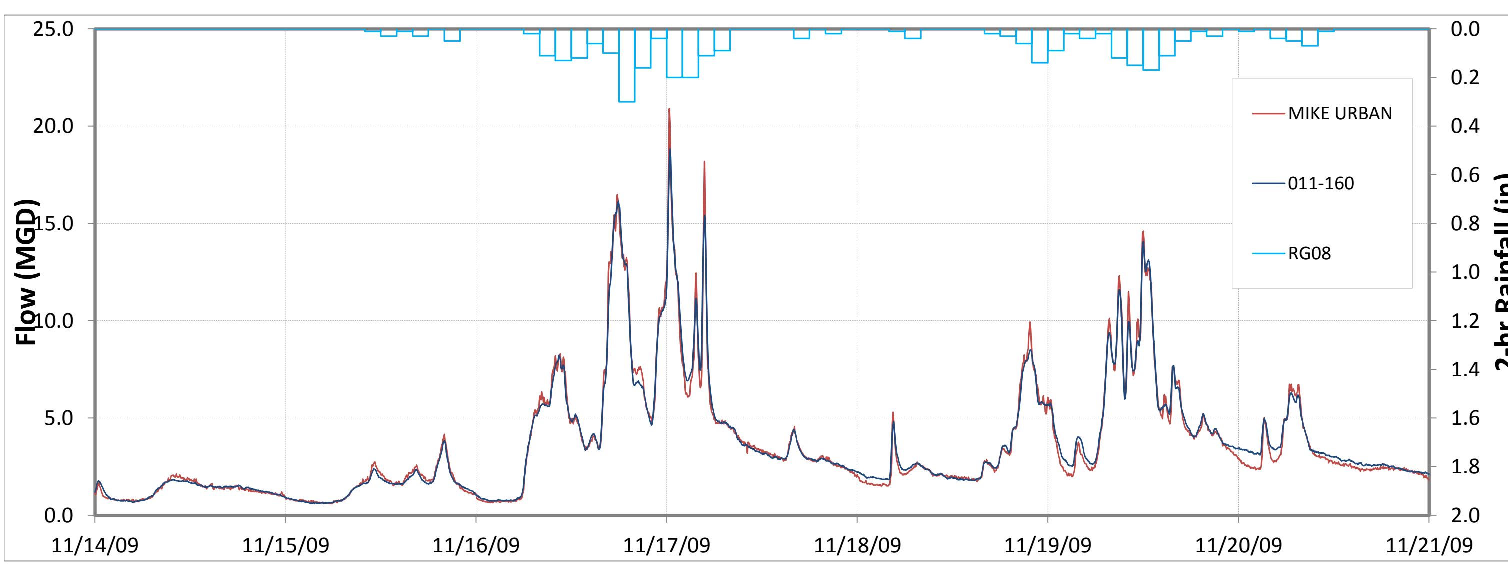


Event 03

11/03/09 to 11/13/09

Parameters	
rdii_area	45.6 %
b_a_iflat	7.6 %
b_m_iflat	0.079
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.04
RMSE	0.46
Nash	0.97
Q_{pk} Error	3.13%
V_{tot} Error	4.01%

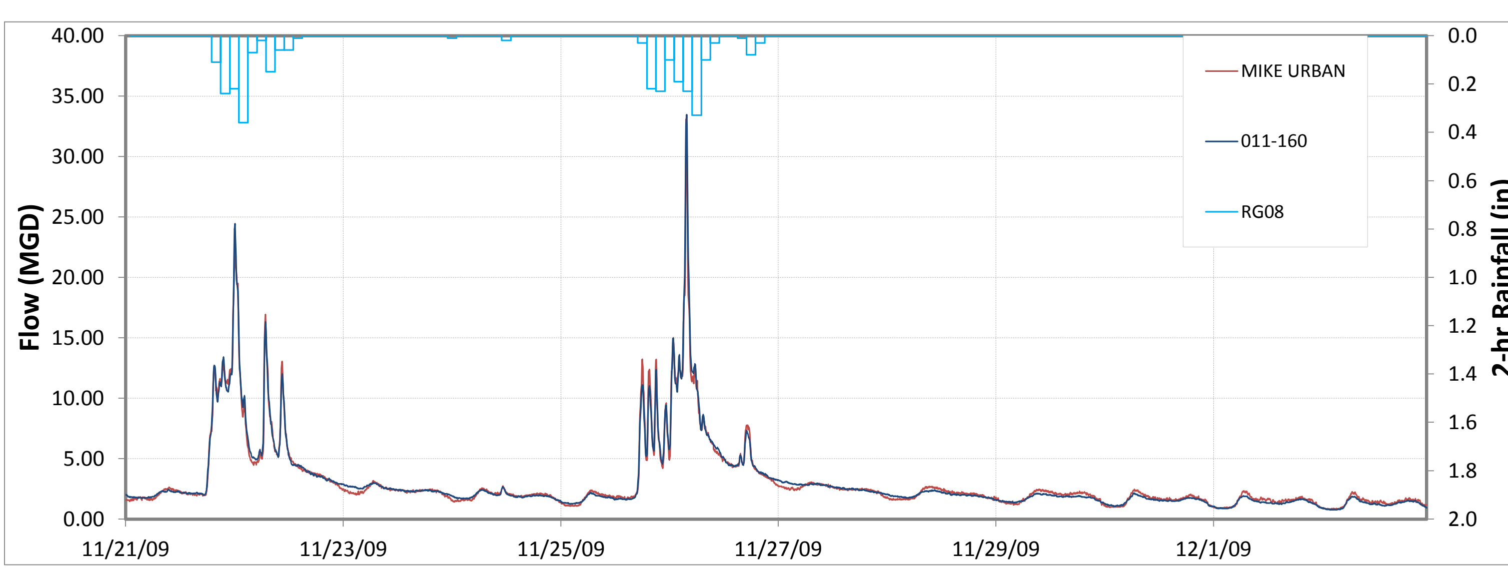


Event 04

11/14/09 to 11/21/09

Parameters	
rdii_area	45.6 %
b_a_iflat	7.6 %
b_m_iflat	0.079
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	-0.01
RMSE	0.32
Nash	0.99
Q_{pk} Error	11.02%
V_{tot} Error	0.29%



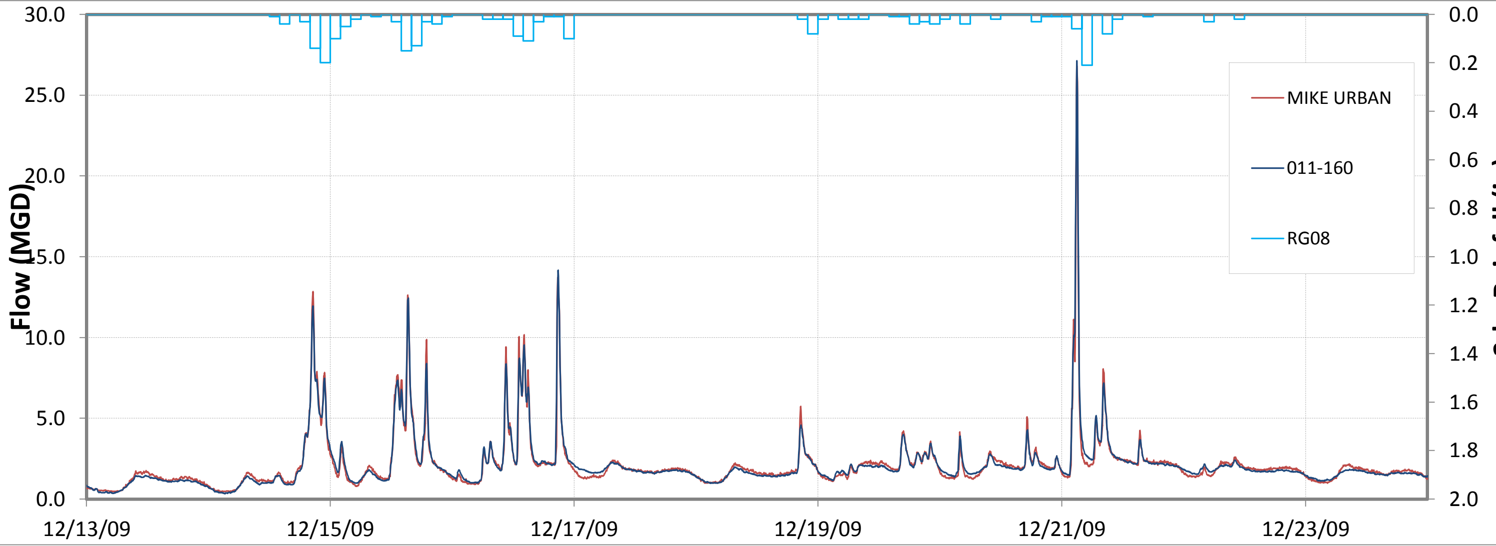
Event 05

11/21/09 to 12/03/09

Parameters	
rdii_area	45.6 %
b_a_iflat	7.6 %
b_m_iflat	0.079
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.00
RMSE	0.30
Nash	0.99
Q_{pk} Error	-0.43%
V_{tot} Error	-1.24%

SB012

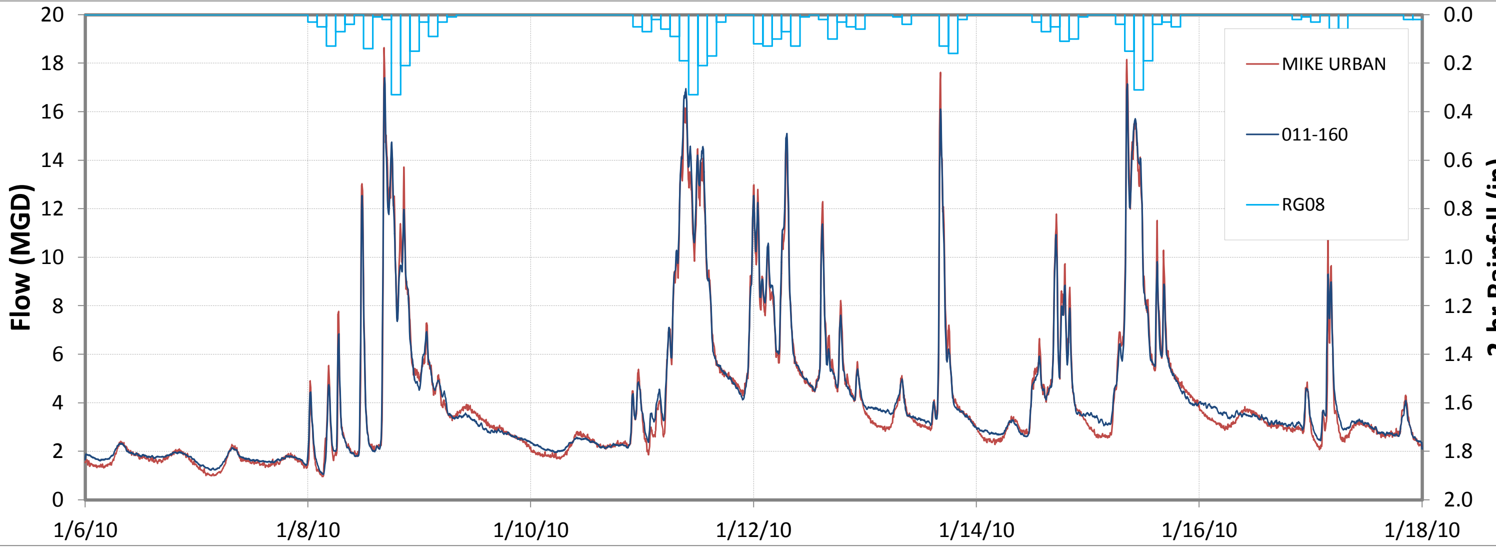


Event 06

12/13/09 to 12/24/09

Parameters	
rdii_area	45.6 %
b_a_iflat	7.6 %
b_m_iflat	0.079
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.01
RMSE	0.25
Nash	0.98
Q_{pk} Error	-1.18%
V_{tot} Error	-2.08%

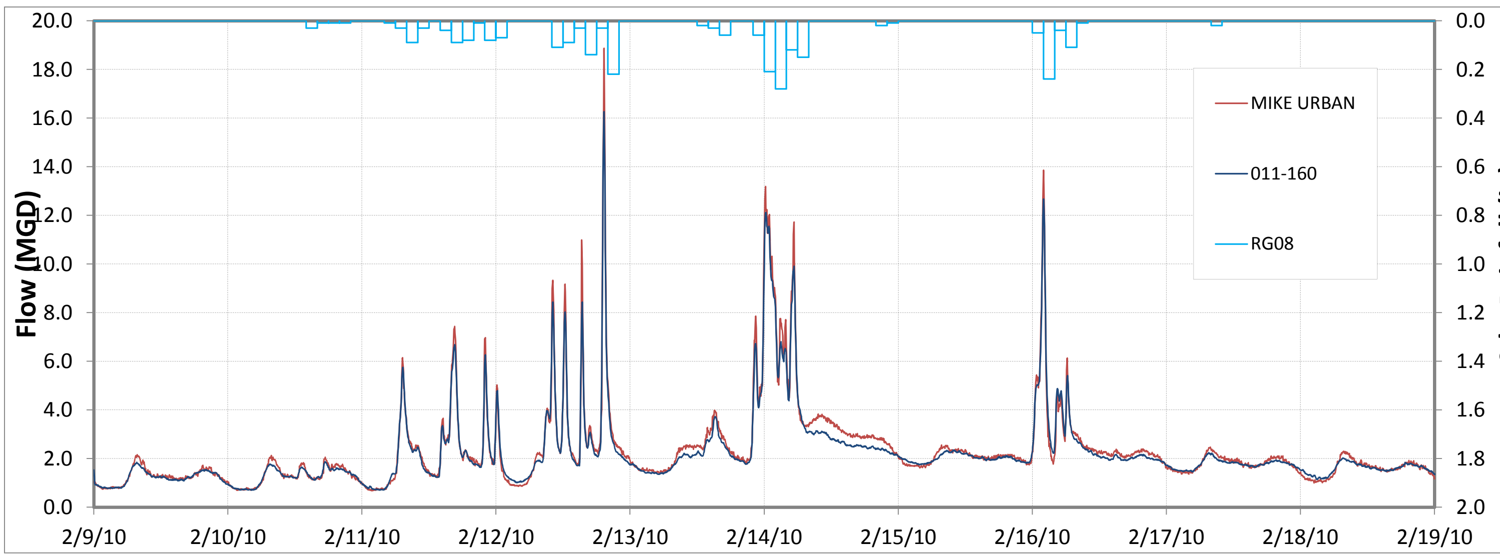


Event 07

01/06/10 to 01/18/10

Parameters	
rdii_area	45.6 %
b_a_iflat	7.6 %
b_m_iflat	0.079
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	-0.02
RMSE	0.35
Nash	0.99
Q_{pk} Error	7.12%
V_{tot} Error	1.72%

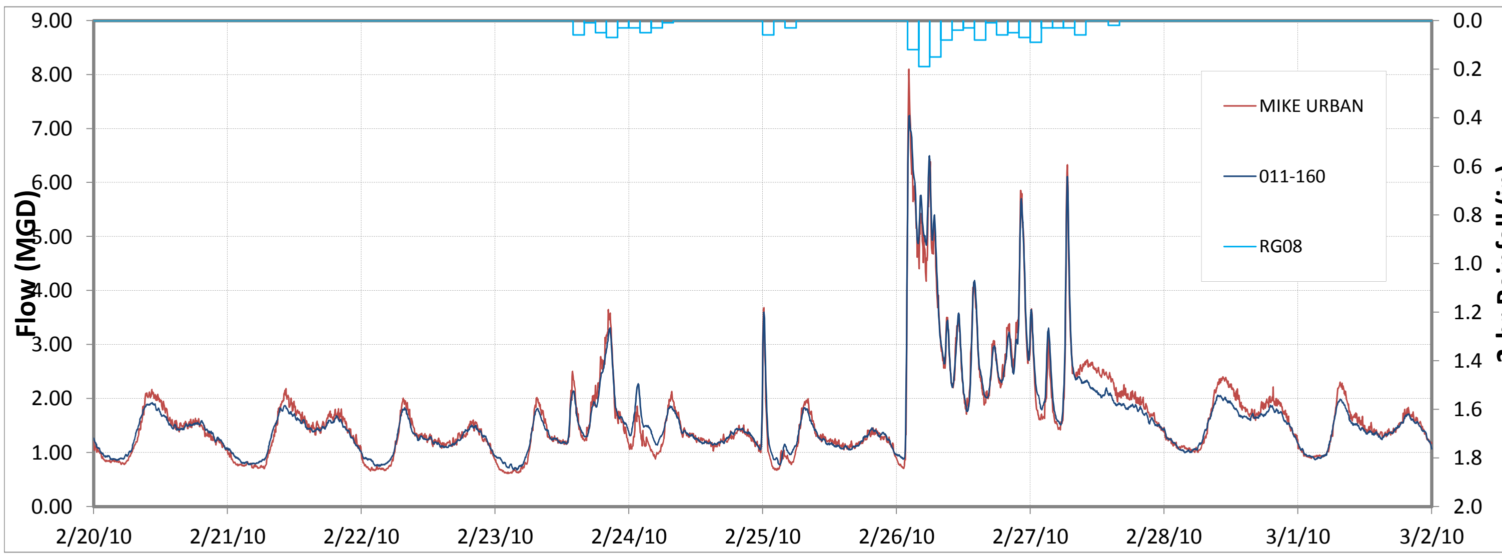


Event 08

02/09/10 to 02/19/10

Parameters	
rdii_area	45.6 %
b_a_iflat	7.6 %
b_m_iflat	0.079
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.04
RMSE	0.25
Nash	0.97
Q_{pk} Error	16.03%
V_{tot} Error	4.89%

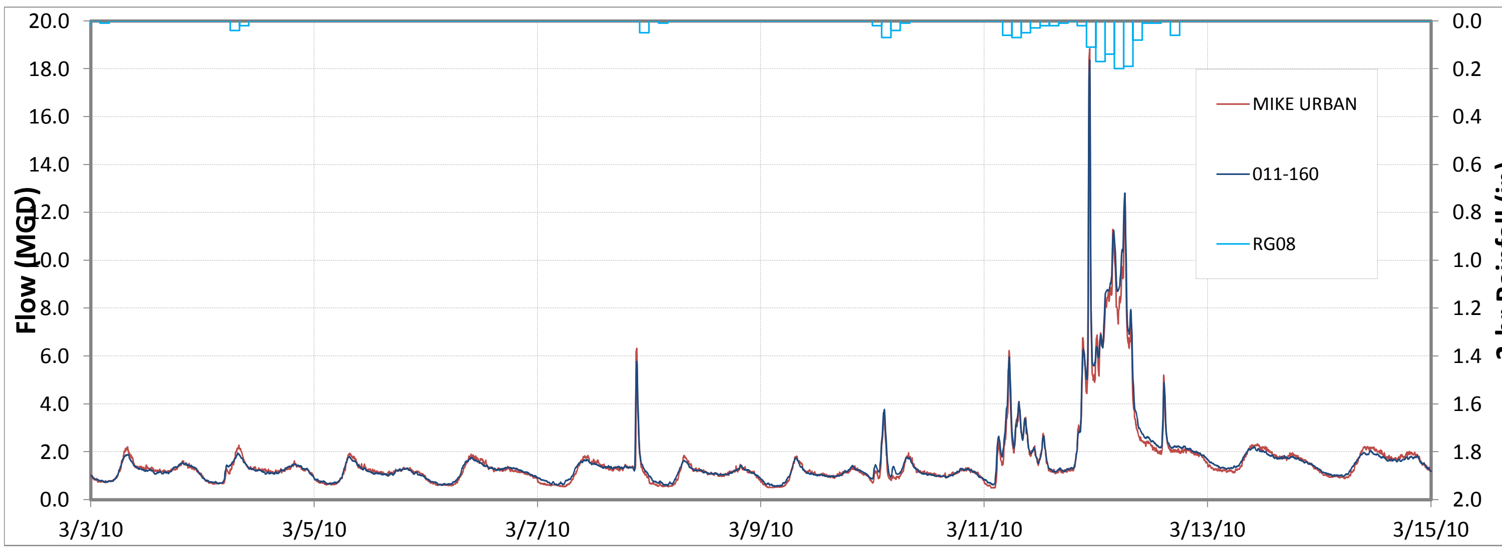


Event 09

02/20/10 to 03/02/10

Parameters	
rdii_area	45.6 %
b_a_iflat	7.6 %
b_m_iflat	0.079
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.01
RMSE	0.16
Nash	0.97
Q_{pk} Error	11.87%
V_{tot} Error	-1.74%



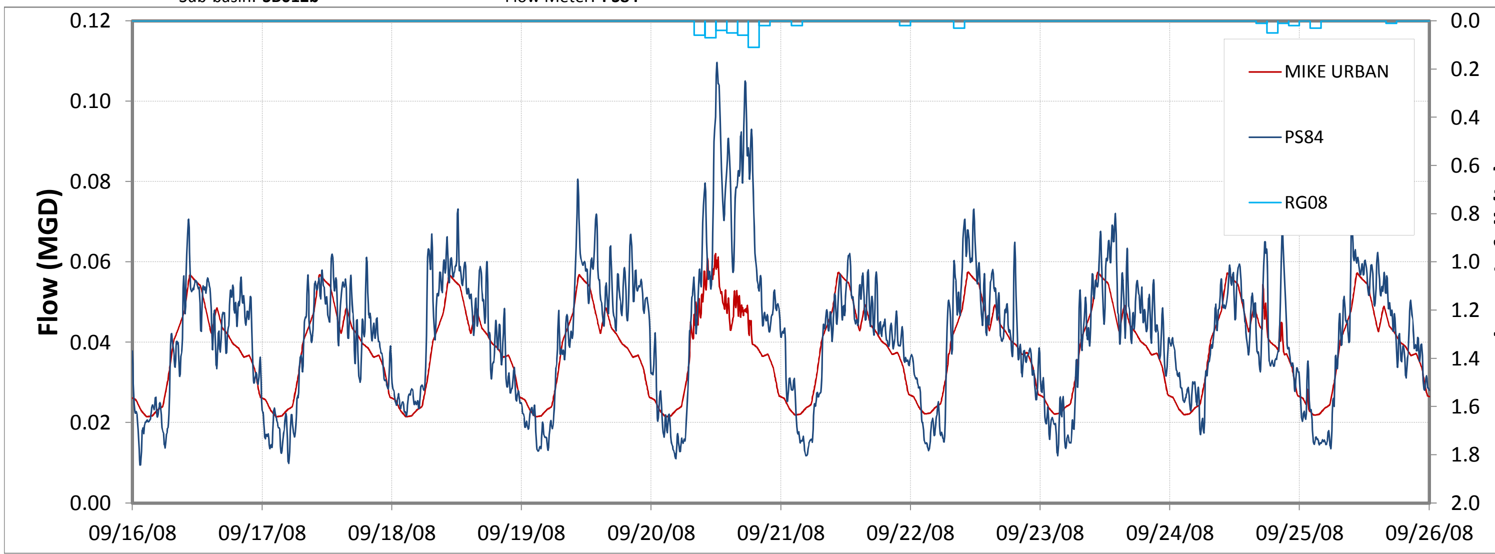
Event 10

03/03/10 to 03/15/10

Parameters	
rdii_area	45.6 %
b_a_iflat	7.6 %
b_m_iflat	0.079
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	-0.03
RMSE	0.19
Nash	0.98
Q_{pk} Error	2.55%
V_{tot} Error	-5.99%

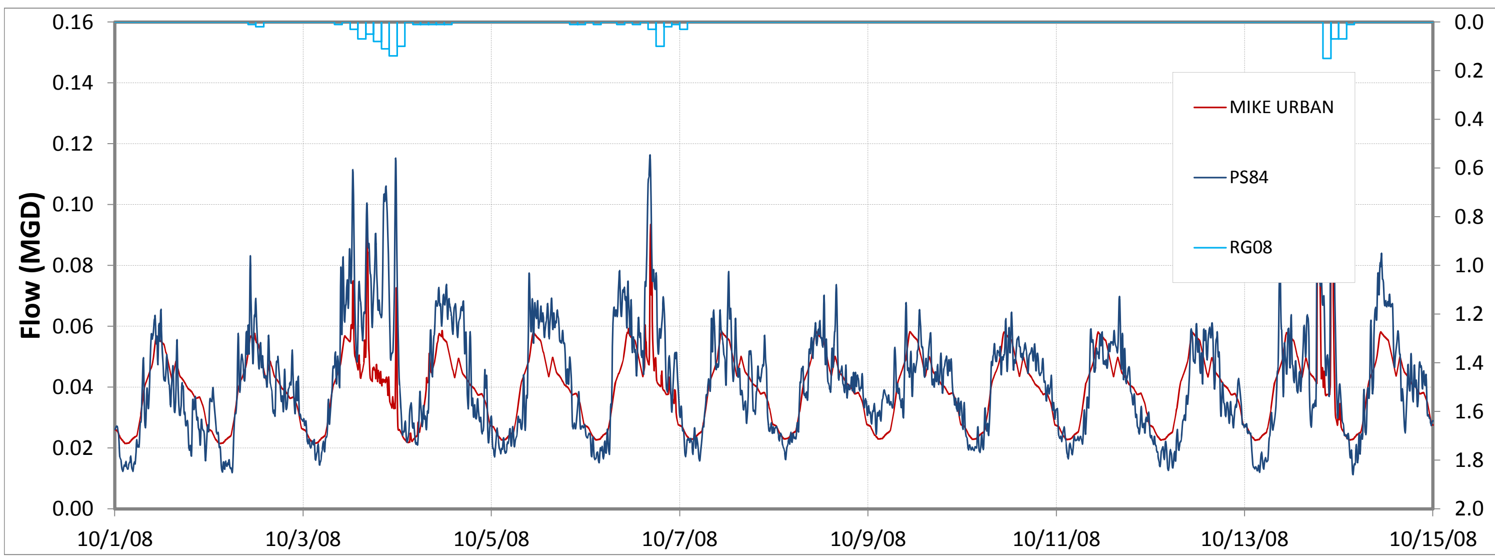
Sub-basin: **SB012b** Flow Meter: **PS84**



Event 01
09/16/08 to 09/26/08

Parameters		
rdii_area	26.1	%
b_a_iflat	3.5	%
b_m_iflat	0.035	
umax	1.67	in
lmax	6.14	in
cqof	0.57	
ck	7.9	hr
ckif	193.9	hr
ckbf	998.2	hr
i_u	0.39	in
i_l	4.43	in
gw_carea	1.000	

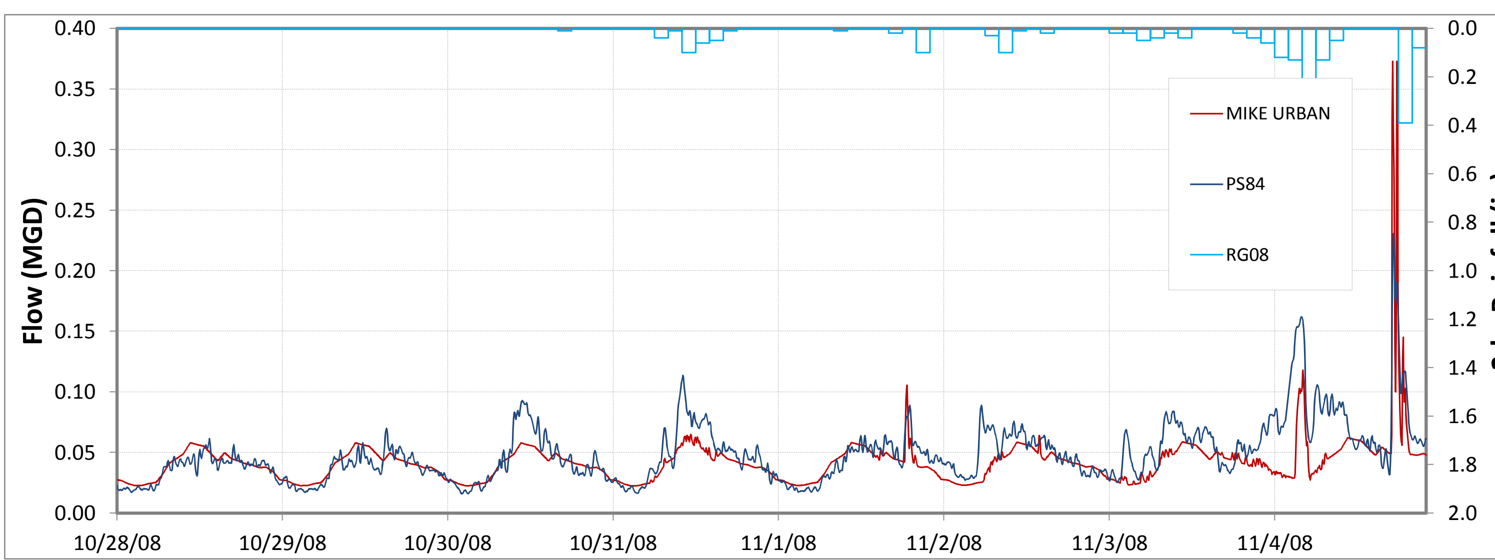
Goodness-of-Fit	
Bias	-0.09
RMSE	0.01
Nash	0.60
Q_{pk} Error	-43.37%
V_{tot} Error	-20.34%



Event 02
10/01/08 to 10/15/08

Parameters		
rdii_area	26.1	%
b_a_iflat	3.5	%
b_m_iflat	0.035	
umax	1.67	in
lmax	6.14	in
cqof	0.57	
ck	7.9	hr
ckif	193.9	hr
ckbf	998.2	hr
i_u	0.39	in
i_l	4.43	in
gw_carea	1.000	

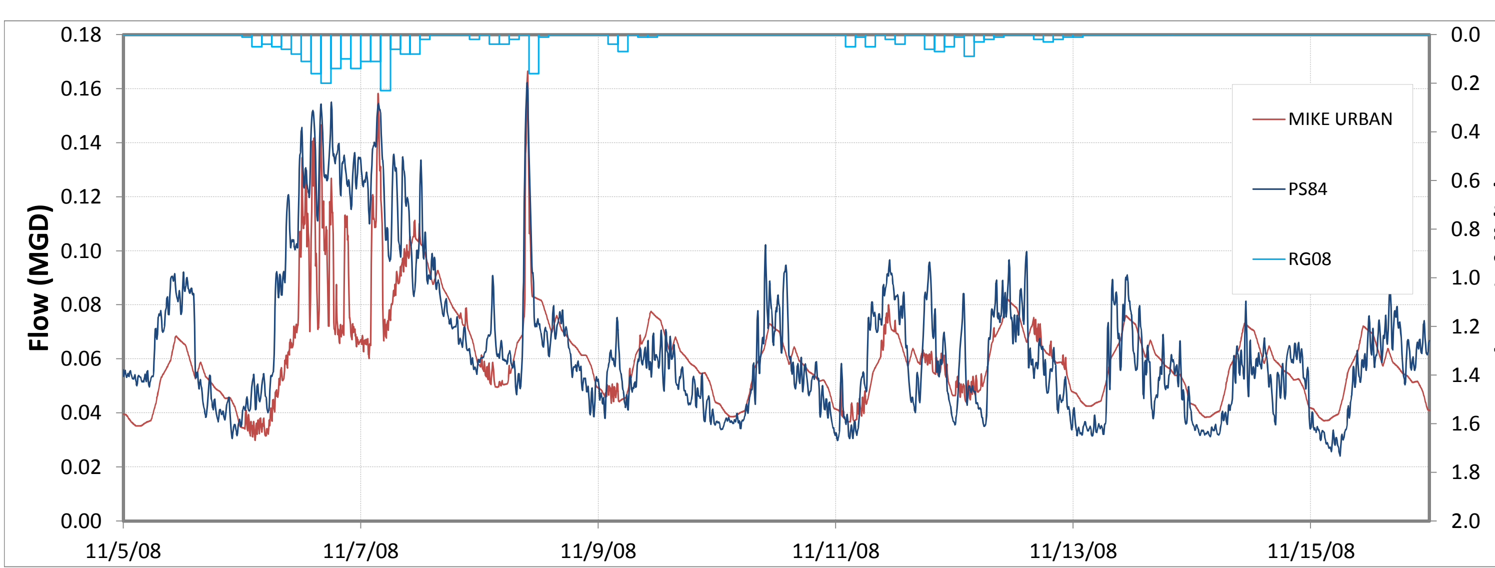
Goodness-of-Fit	
Bias	-0.06
RMSE	0.01
Nash	0.61
Q_{pk} Error	-9.28%
V_{tot} Error	-3.79%



Event 03
10/28/08 to 11/04/08

Parameters		
rdii_area	26.1	%
b_a_iflat	3.5	%
b_m_iflat	0.035	
umax	1.67	in
lmax	6.14	in
cqof	0.57	
ck	7.9	hr
ckif	193.9	hr
ckbf	998.2	hr
i_u	0.39	in
i_l	4.43	in
gw_carea	1.000	

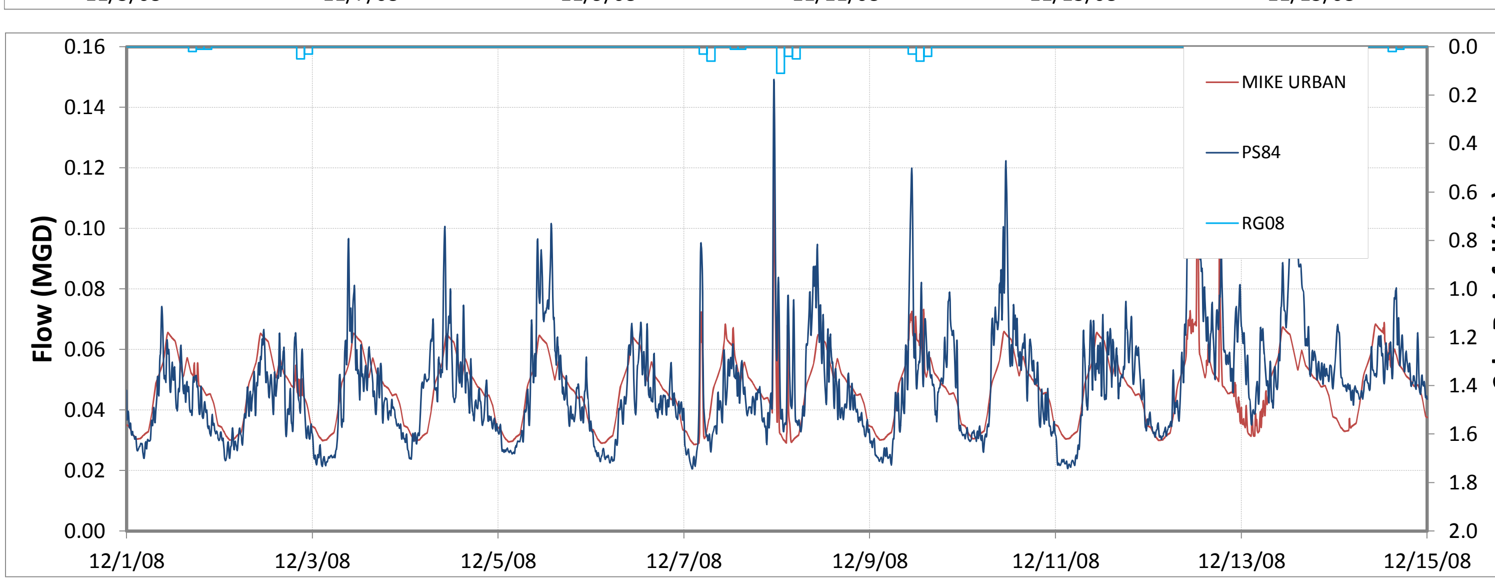
Goodness-of-Fit	
Bias	-0.14
RMSE	0.02
Nash	0.39
Q_{pk} Error	61.70%
V_{tot} Error	-27.22%



Event 04
11/05/08 to 11/16/08

Parameters		
rdii_area	26.1	%
b_a_iflat	3.5	%
b_m_iflat	0.035	
umax	1.67	in
lmax	6.14	in
cqof	0.57	
ck	7.9	hr
ckif	193.9	hr
ckbf	998.2	hr
i_u	0.39	in
i_l	4.43	in
gw_carea	1.000	

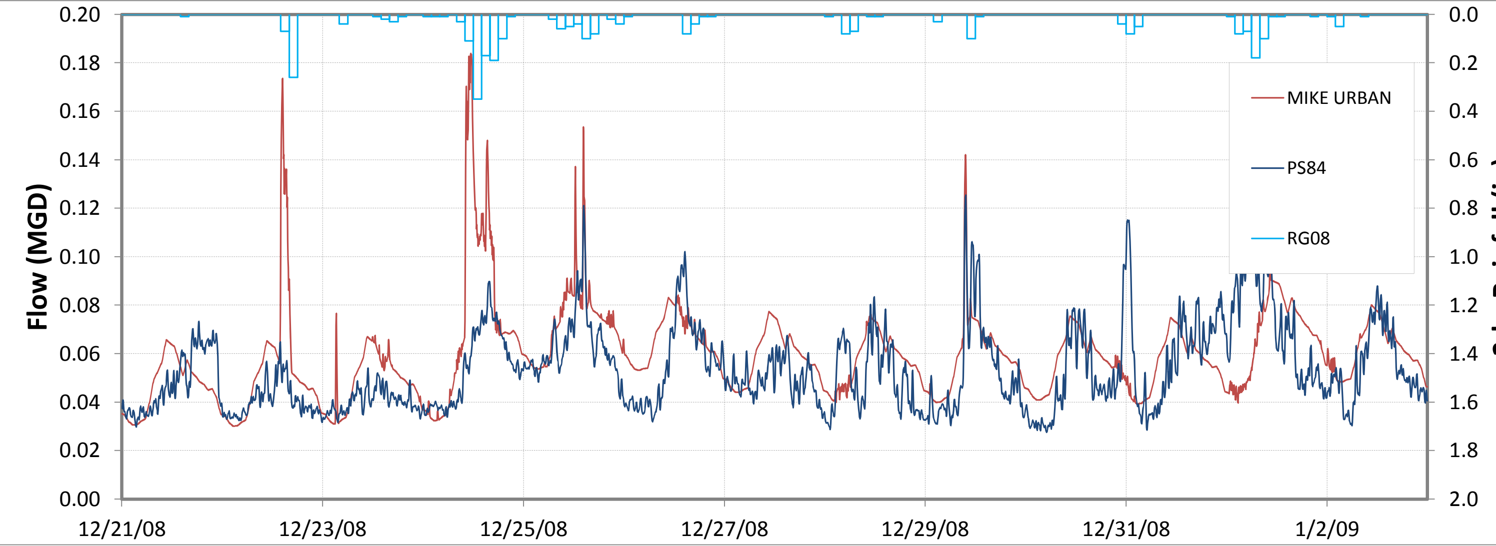
Goodness-of-Fit	
Bias	-0.06
RMSE	0.02
Nash	0.59
Q_{pk} Error	2.60%
V_{tot} Error	1.19%



Event 05
12/01/08 to 12/15/08

Parameters		
rdii_area	26.1	%
b_a_iflat	3.5	%
b_m_iflat	0.035	
umax	1.67	in
lmax	6.14	in
cqof	0.57	
ck	7.9	hr
ckif	193.9	hr
ckbf	998.2	hr
i_u	0.39	in
i_l	4.43	in
gw_carea	1.000	

Goodness-of-Fit	
Bias	-0.03
RMSE	0.01
Nash	0.52
Q_{pk} Error	-4.30%
V_{tot} Error	0.85%

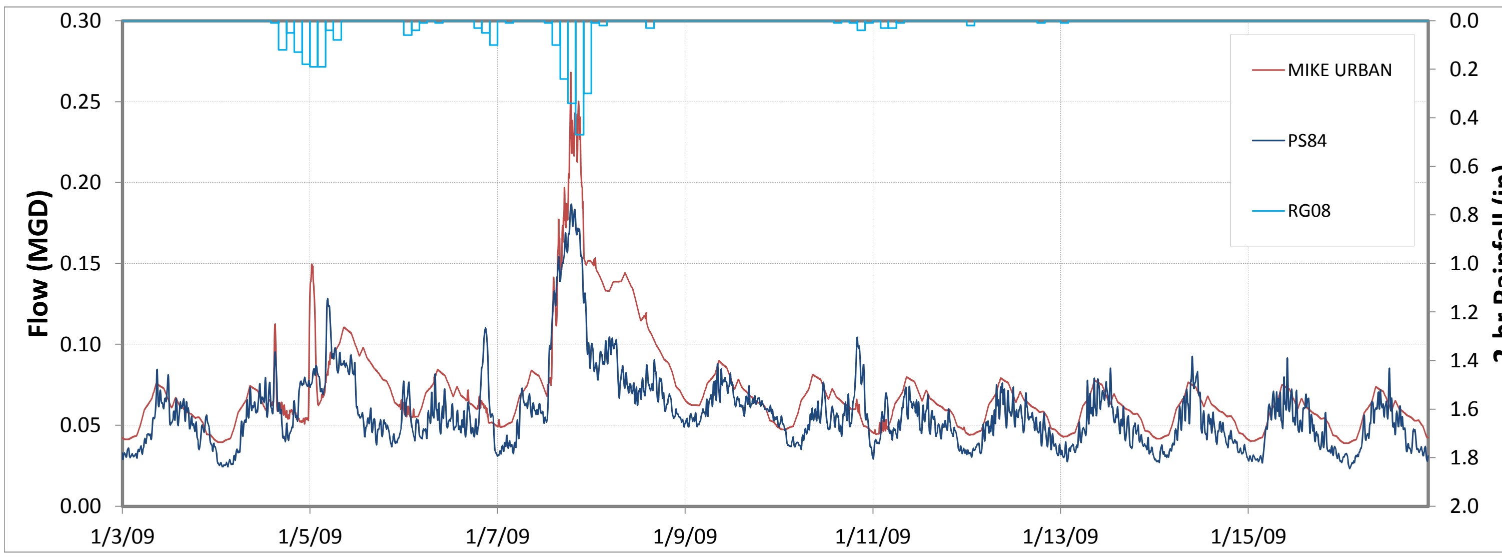


Event 06

12/21/08 to 01/03/09

Parameters	
rdii_area	26.1 %
b_a_iflat	3.5 %
b_m_iflat	0.035
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.12
RMSE	0.02
Nash	-0.27
Q_{pk} Error	36.74%
V_{tot} Error	27.83%

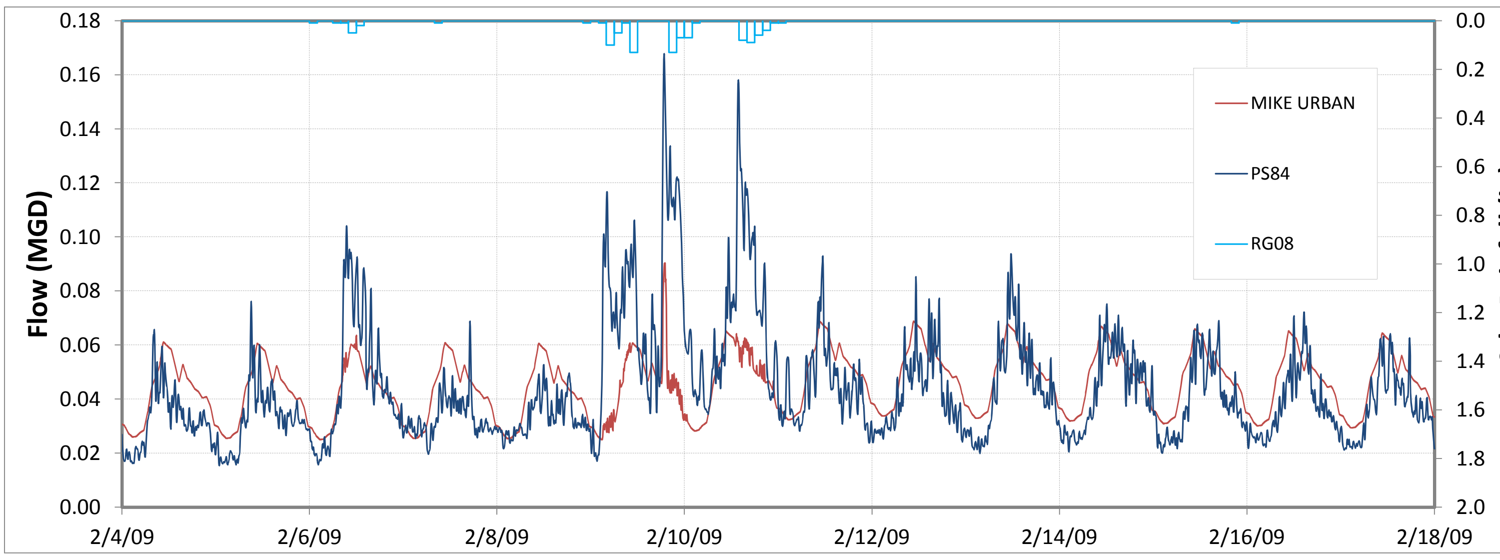


Event 07

01/03/09 to 01/17/09

Parameters	
rdii_area	26.1 %
b_a_iflat	3.5 %
b_m_iflat	0.035
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.22
RMSE	0.02
Nash	0.25
Q_{pk} Error	43.69%
V_{tot} Error	32.14%

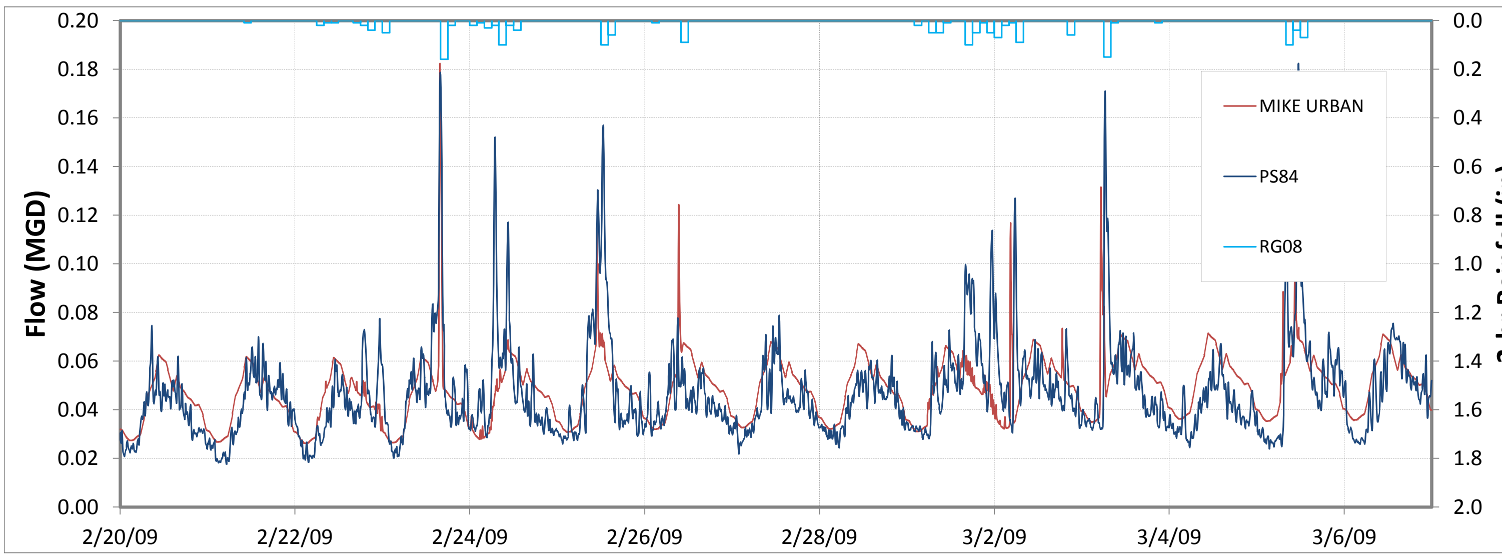


Event 08

02/04/09 to 02/18/09

Parameters	
rdii_area	26.1 %
b_a_iflat	3.5 %
b_m_iflat	0.035
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.03
RMSE	0.02
Nash	0.33
Q_{pk} Error	-46.17%
V_{tot} Error	-34.43%

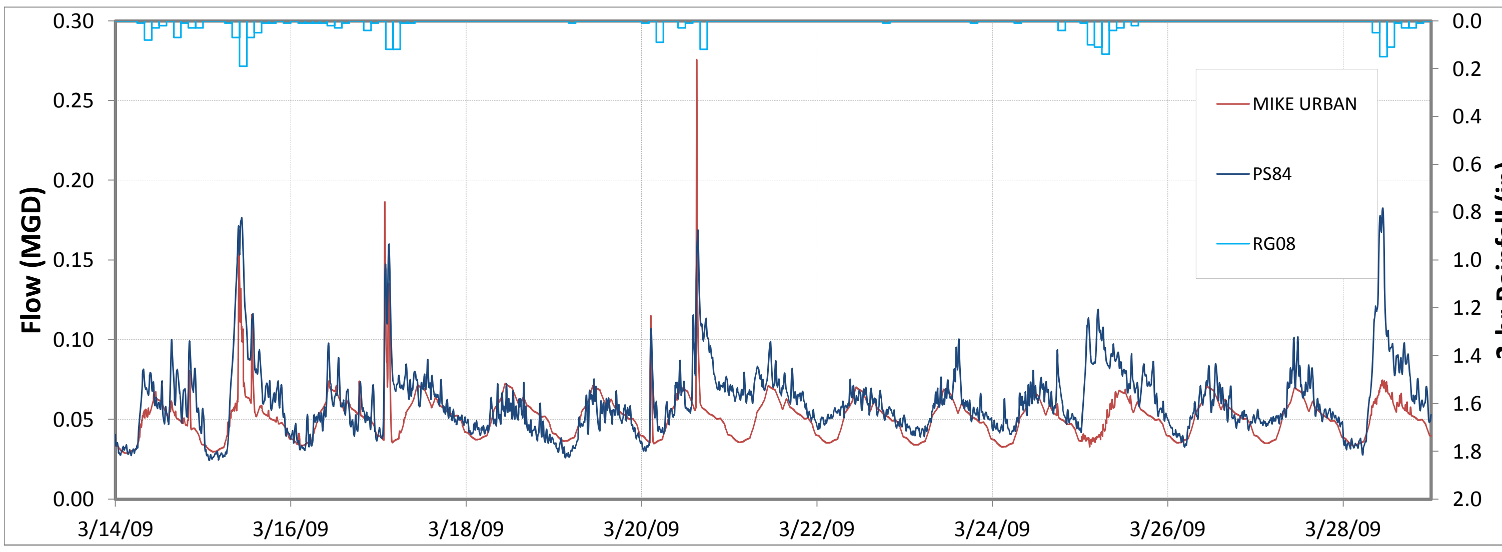


Event 09

02/20/09 to 03/07/09

Parameters	
rdii_area	26.1 %
b_a_iflat	3.5 %
b_m_iflat	0.035
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.04
RMSE	0.02
Nash	0.31
Q_{pk} Error	0.05%
V_{tot} Error	-10.92%



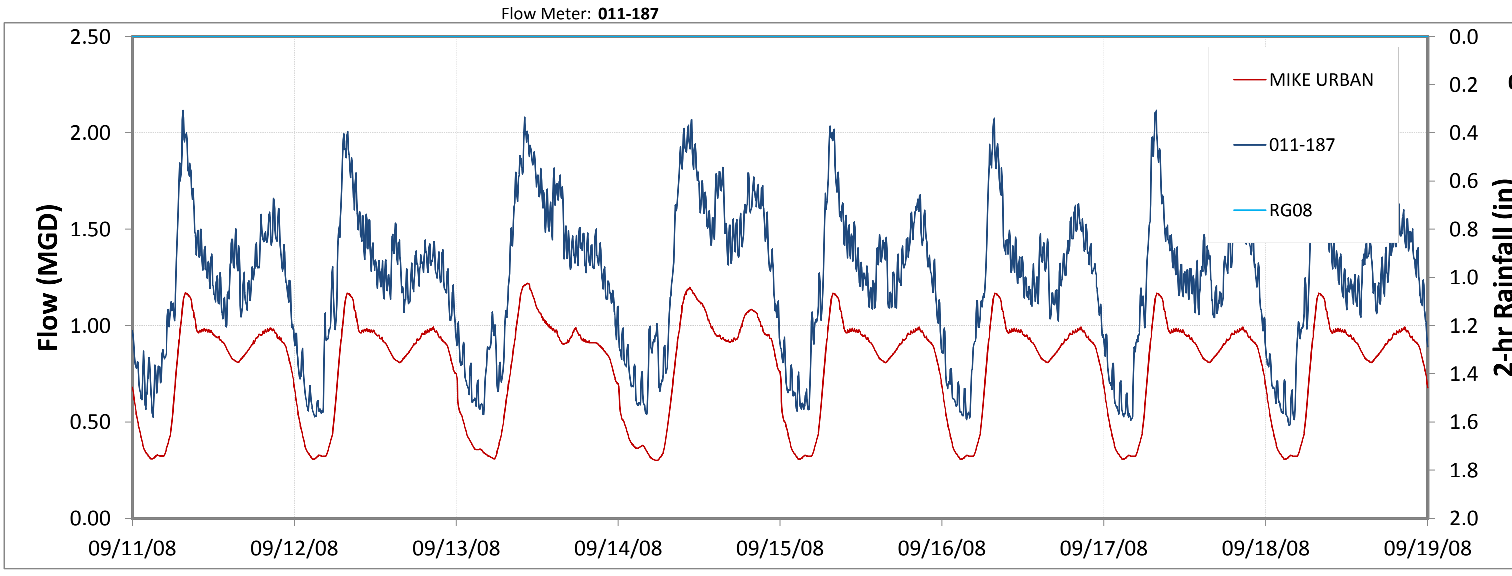
Event 10

03/14/09 to 03/29/09

Parameters	
rdii_area	26.1 %
b_a_iflat	3.5 %
b_m_iflat	0.035
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

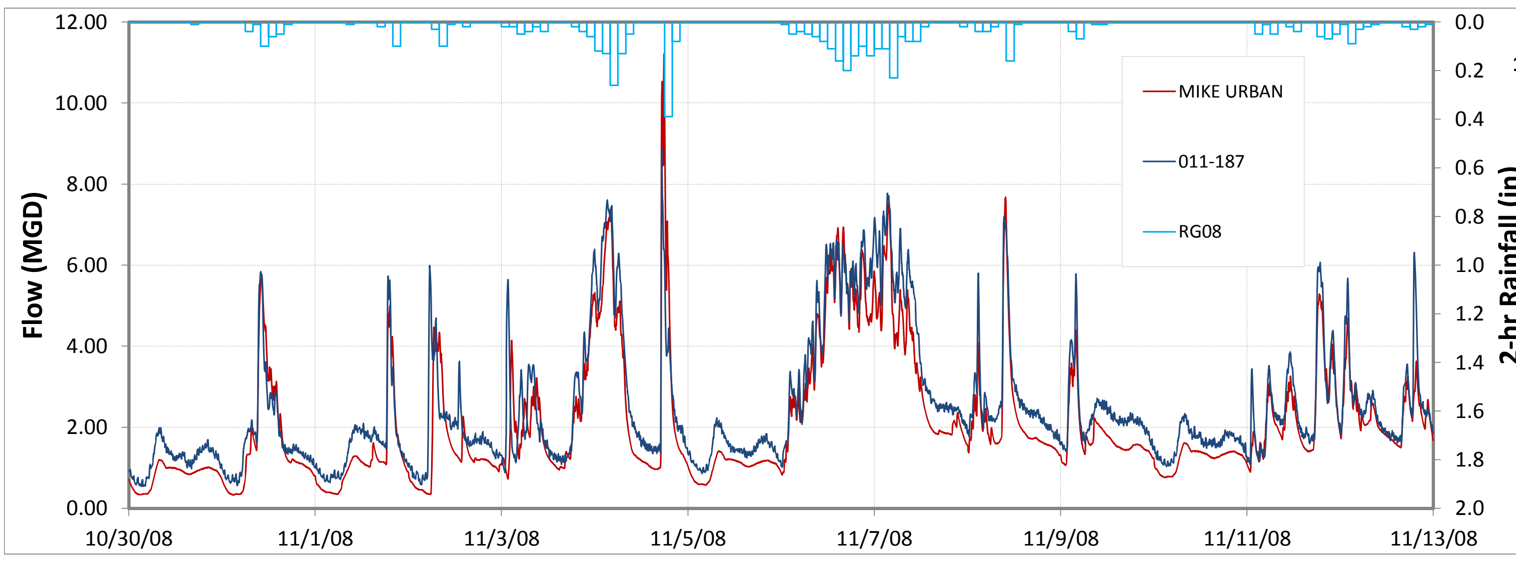
Goodness-of-Fit	
Bias	-0.16
RMSE	0.02
Nash	0.22
Q_{pk} Error	50.82%
V_{tot} Error	-20.74%

Flow Meter: **011-187**



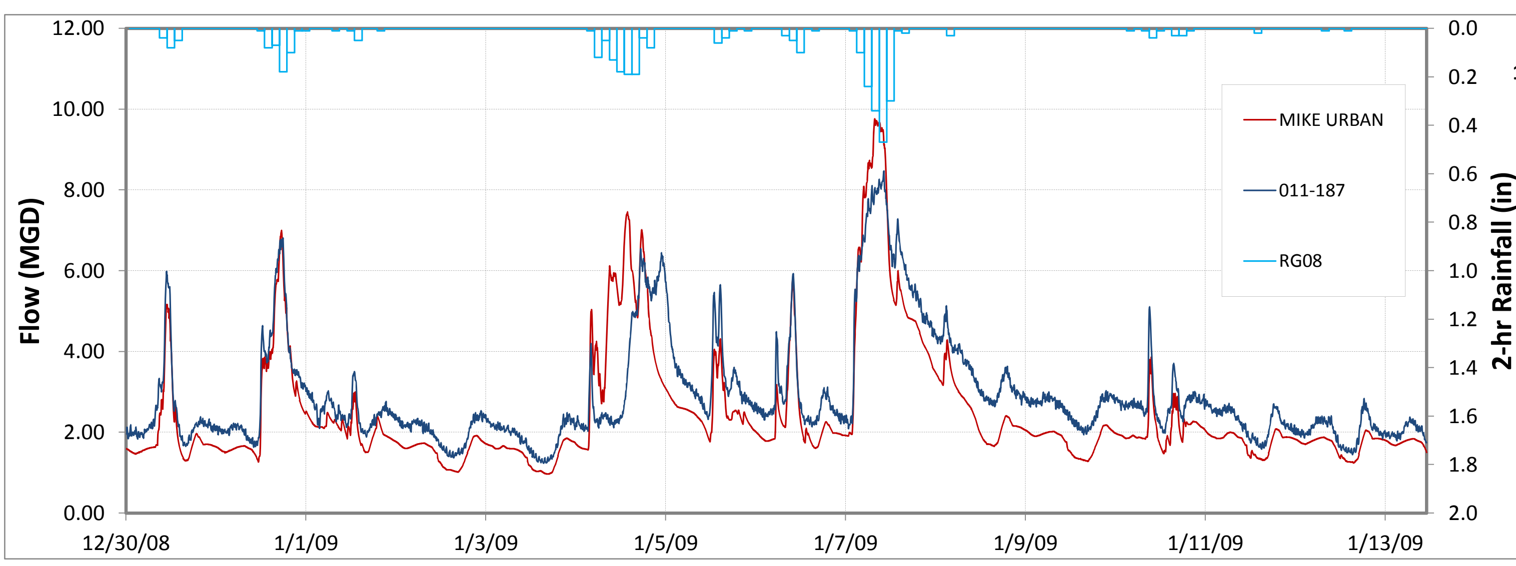
Event 01
09/11/08 to 09/19/08

Goodness-of-Fit	
Bias	-0.36
RMSE	0.48
Nash	-0.72
Q_{pk} Error	-42.39%
V_{tot} Error	-37.56%



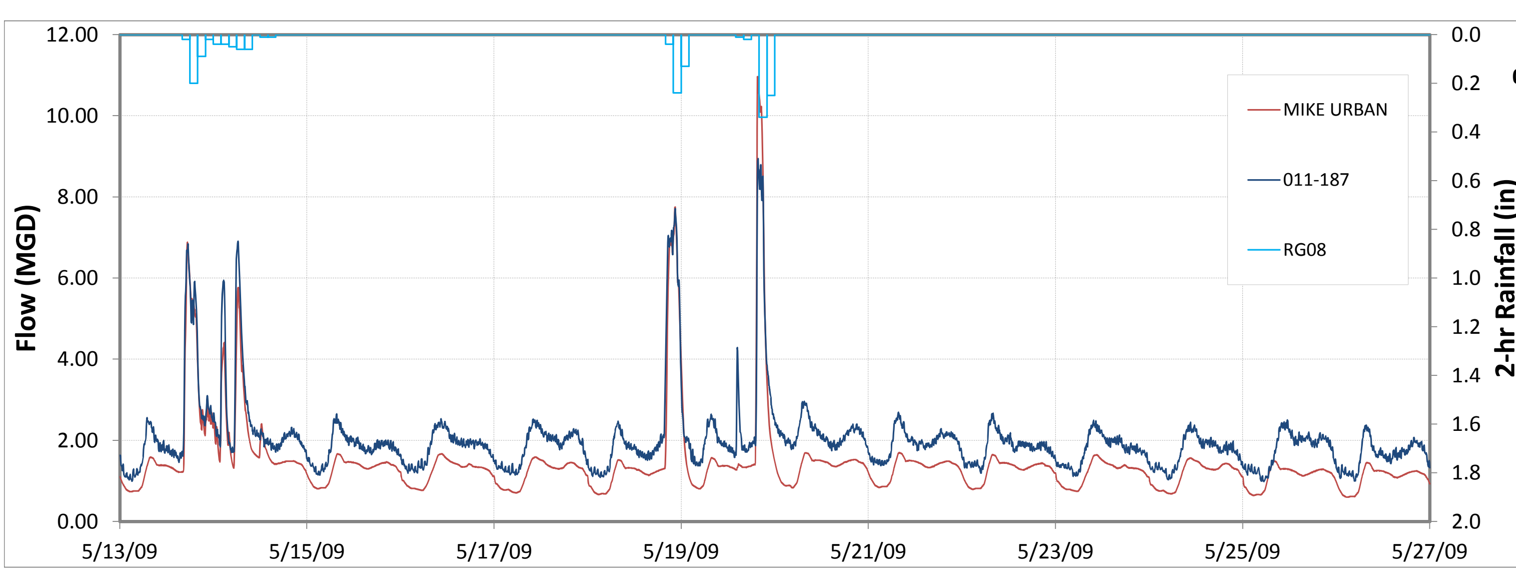
Event 02
10/30/08 to 11/13/08

Goodness-of-Fit	
Bias	-0.16
RMSE	0.69
Nash	0.80
Q_{pk} Error	26.17%
V_{tot} Error	-13.00%



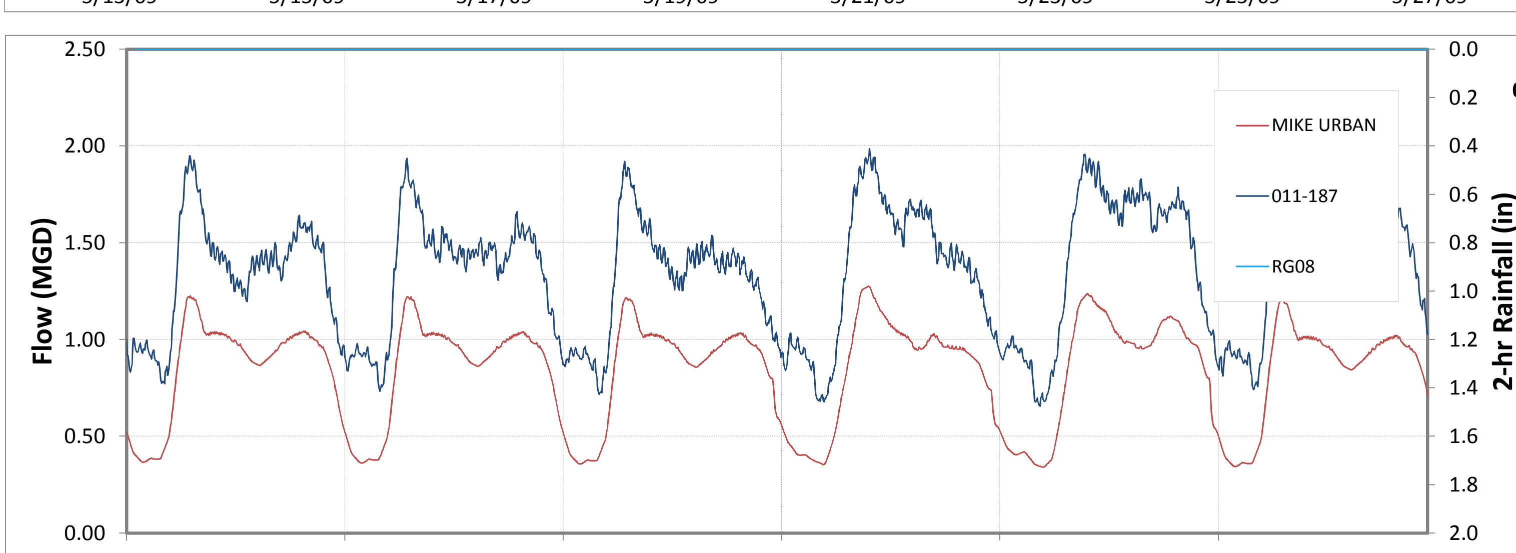
Event 03
12/30/08 to 01/14/09

Goodness-of-Fit	
Bias	-0.15
RMSE	0.84
Nash	0.60
Q_{pk} Error	15.30%
V_{tot} Error	-8.54%



Event 04
05/13/09 to 05/27/09

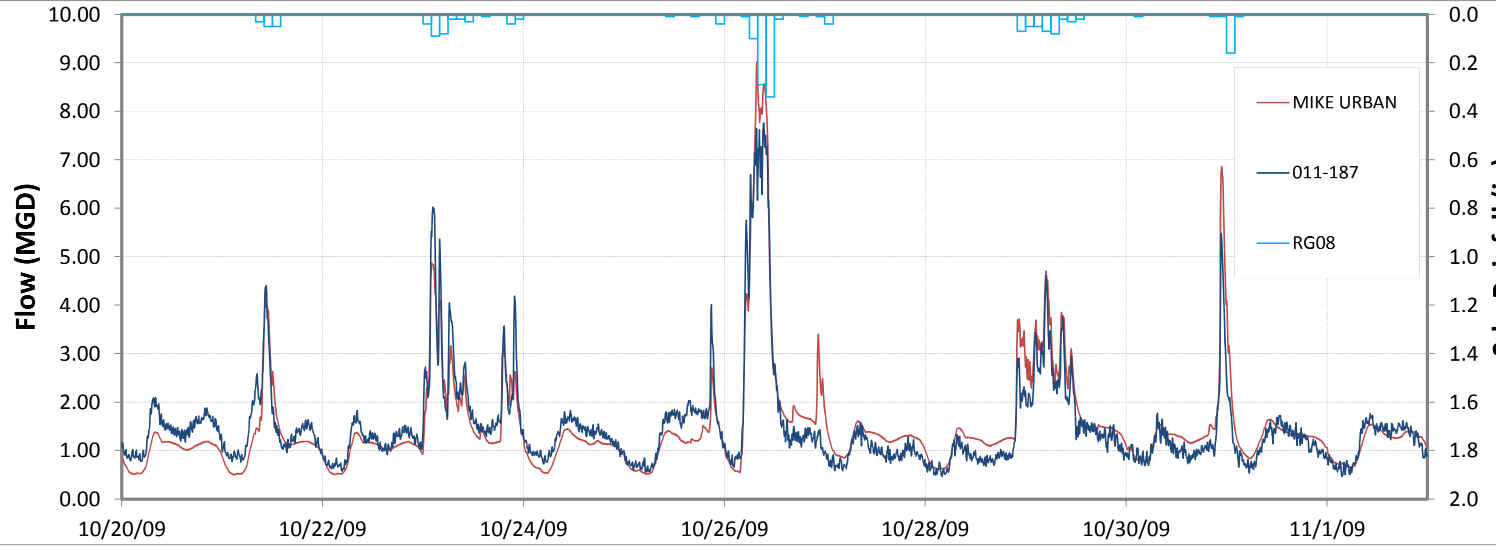
Goodness-of-Fit	
Bias	-0.28
RMSE	0.65
Nash	0.53
Q_{pk} Error	22.58%
V_{tot} Error	-22.80%



Event 05
09/09/09 to 09/15/09

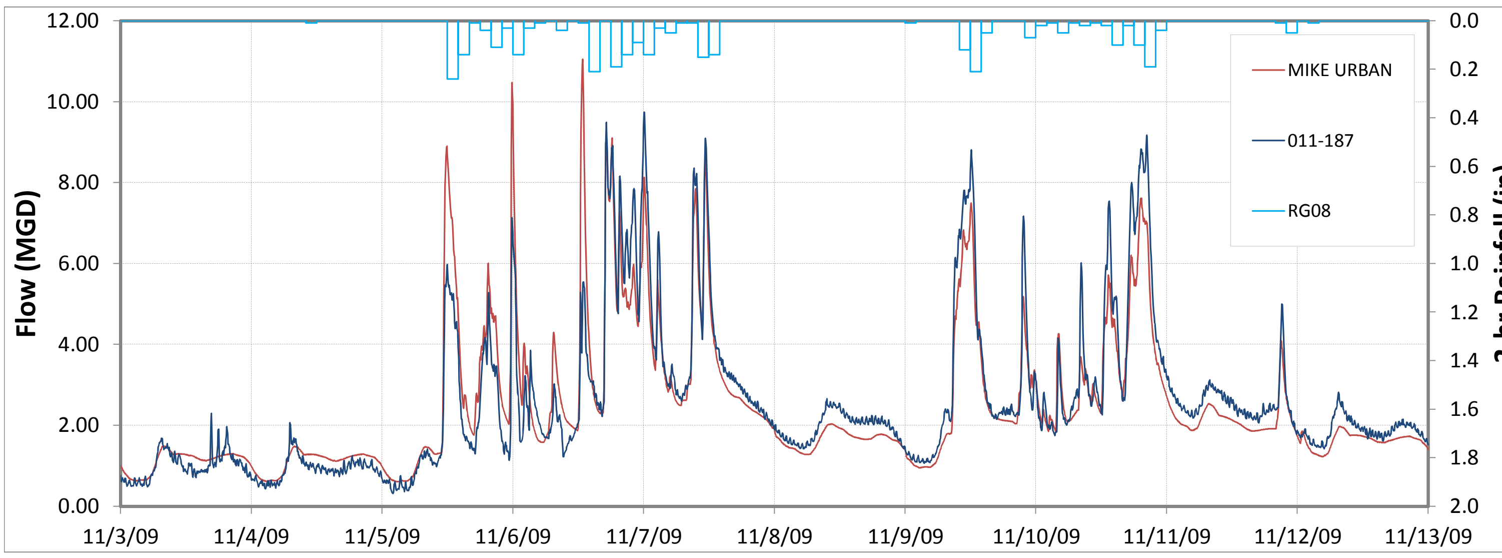
Goodness-of-Fit	
Bias	-0.37
RMSE	0.52
Nash	-1.52
Q_{pk} Error	-35.78%
V_{tot} Error	-37.08%

Flow Meter: **011-187**



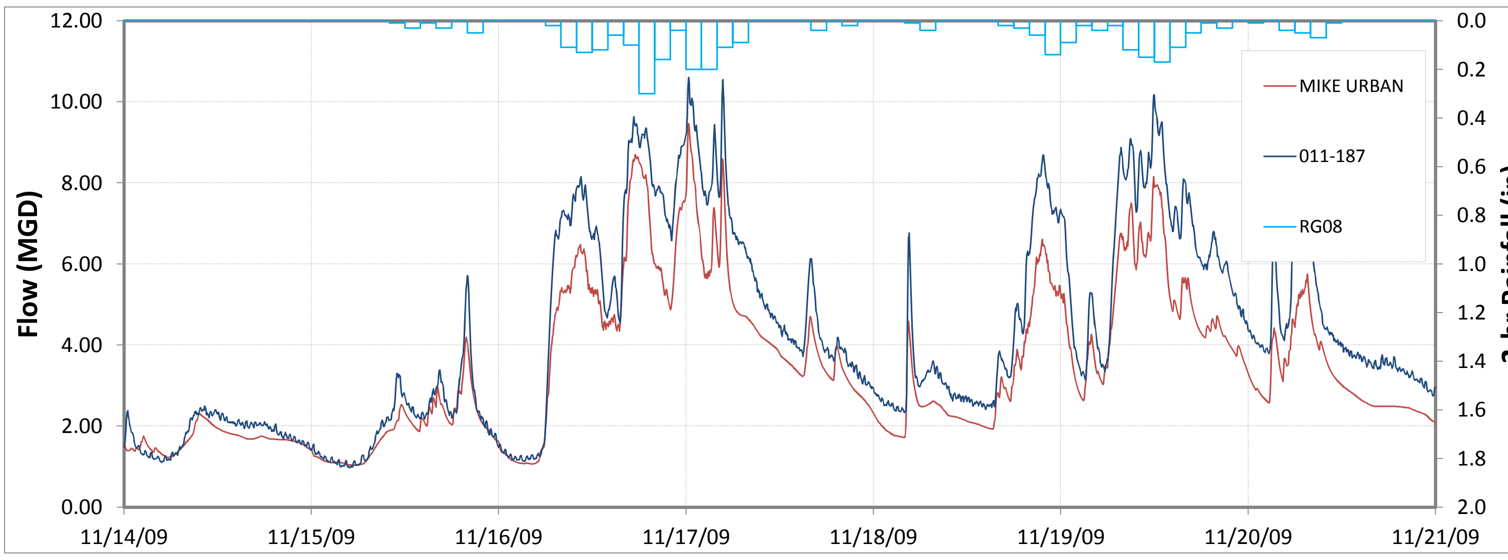
Event 06
10/20/09 to 11/02/09

Goodness-of-Fit	
Bias	0.00
RMSE	0.39
Nash	0.85
Q_{pk} Error	16.25%
V_{tot} Error	2.82%



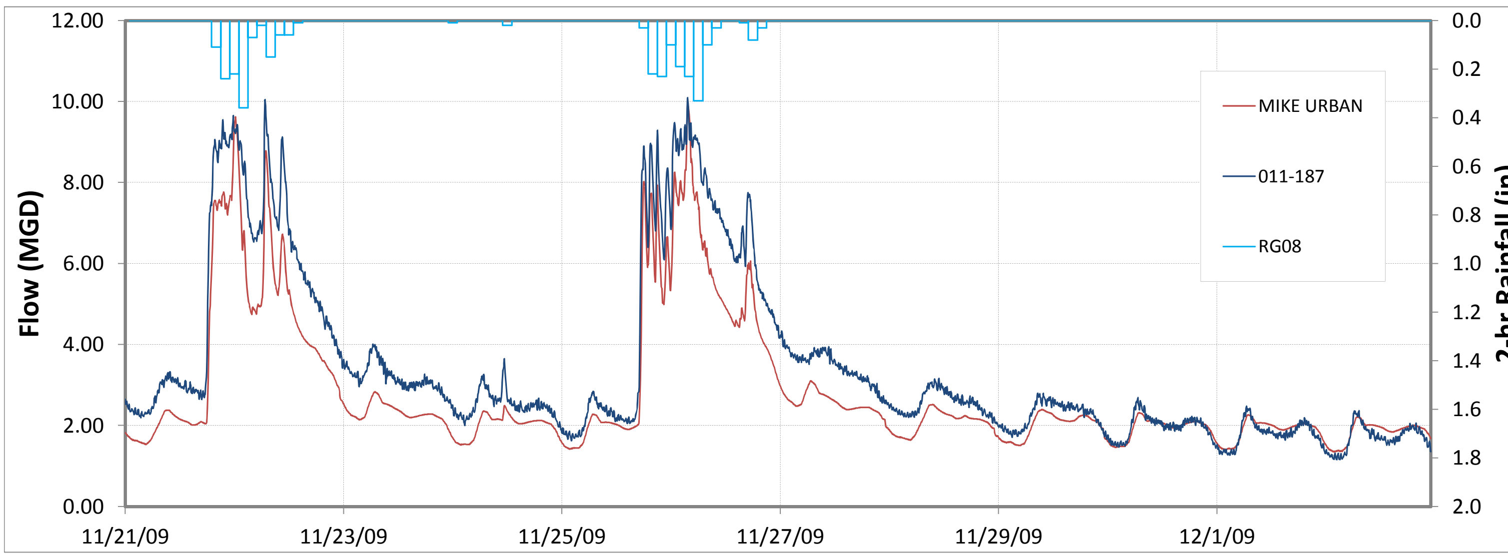
Event 07
11/03/09 to 11/13/09

Goodness-of-Fit	
Bias	-0.04
RMSE	0.67
Nash	0.87
Q_{pk} Error	13.44%
V_{tot} Error	4.99%



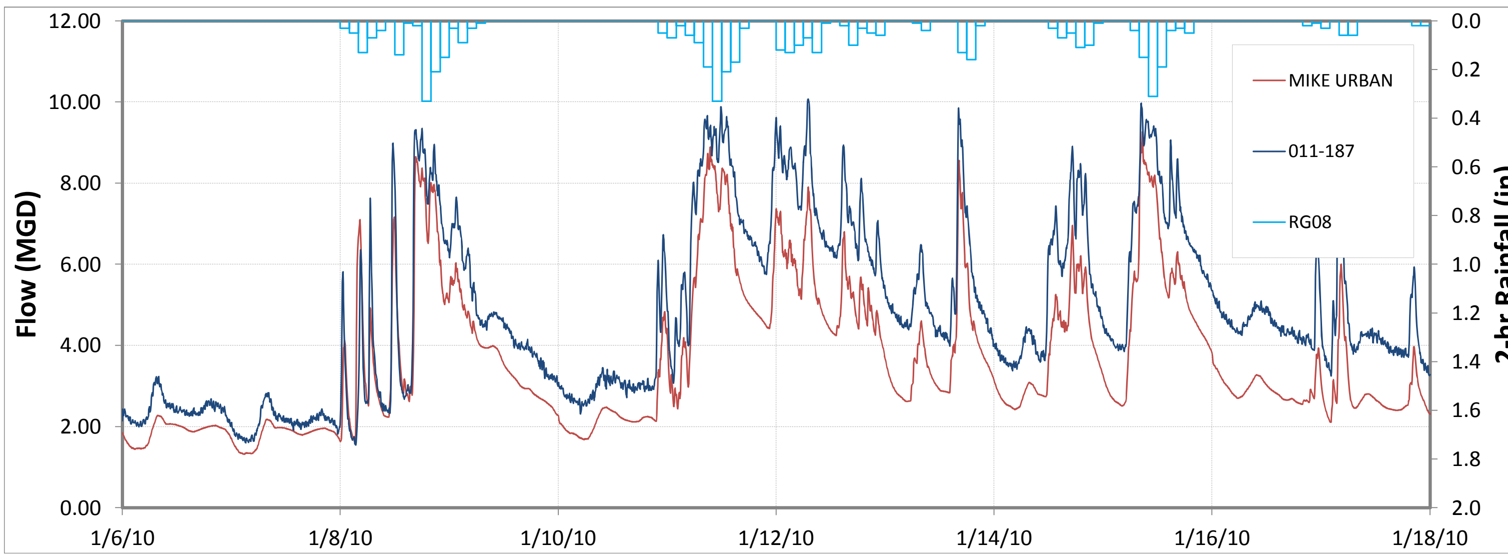
Event 08
11/14/09 to 11/21/09

Goodness-of-Fit	
Bias	-0.20
RMSE	1.09
Nash	0.80
Q_{pk} Error	-10.71%
V_{tot} Error	-18.38%



Event 09
11/21/09 to 12/03/09

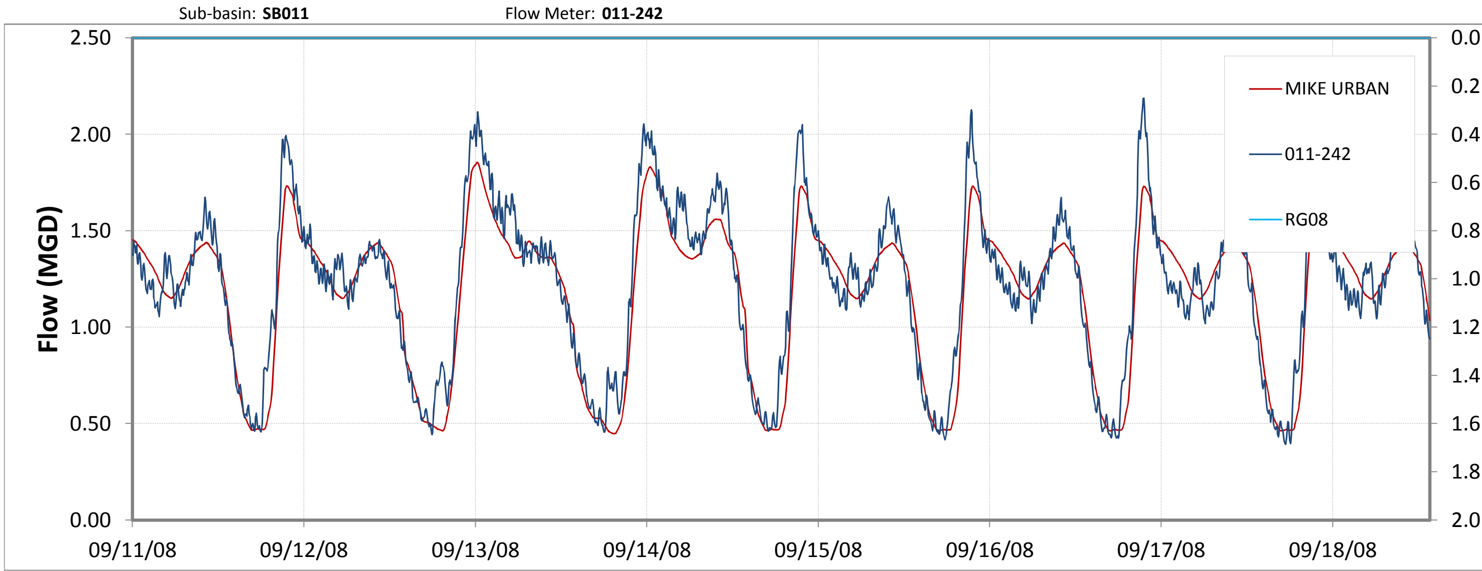
Goodness-of-Fit	
Bias	-0.18
RMSE	0.85
Nash	0.84
Q_{pk} Error	-2.05%
V_{tot} Error	-19.63%



Event 10
01/06/10 to 01/18/10

Goodness-of-Fit	
Bias	-0.24
RMSE	1.37
Nash	0.58
Q_{pk} Error	-8.00%
V_{tot} Error	-25.93%

Sub-basin: **SB011** Flow Meter: **011-242**

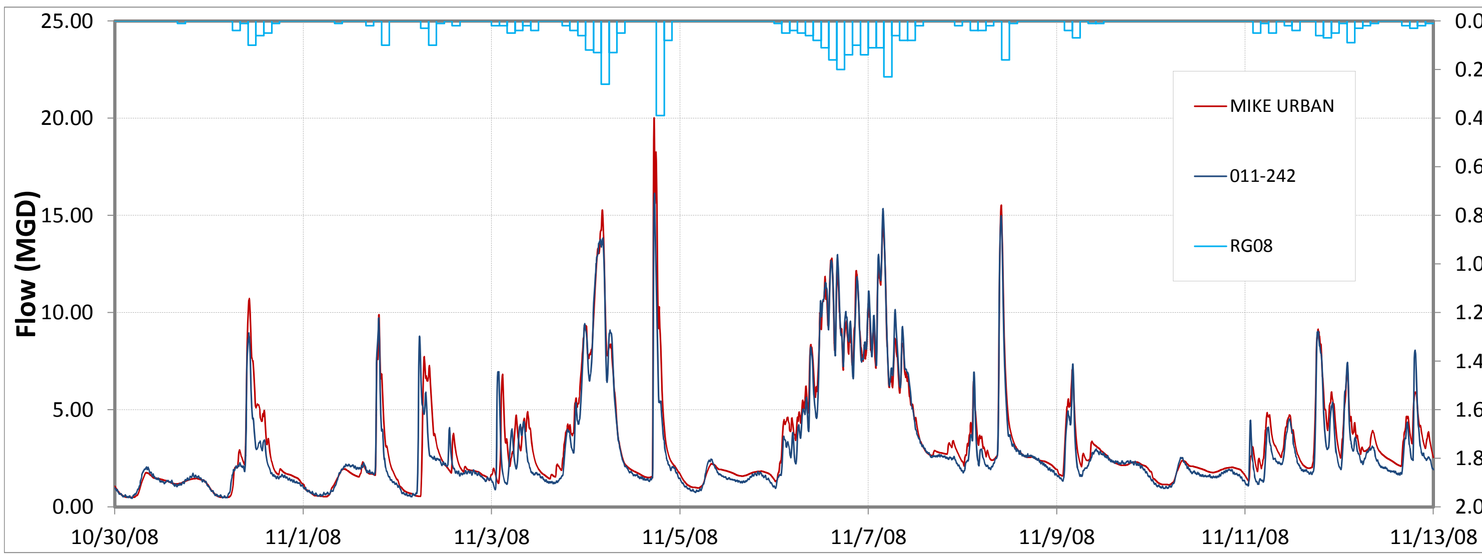


Event 01

09/11/08 to 09/19/08

Parameters	
rdii_area	45.6 %
b_a_iflat	7.6 %
b_m_iflat	0.079
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	-0.03
RMSE	0.14
Nash	0.88
Q_{pk} Error	-15.25%
V_{tot} Error	-5.18%

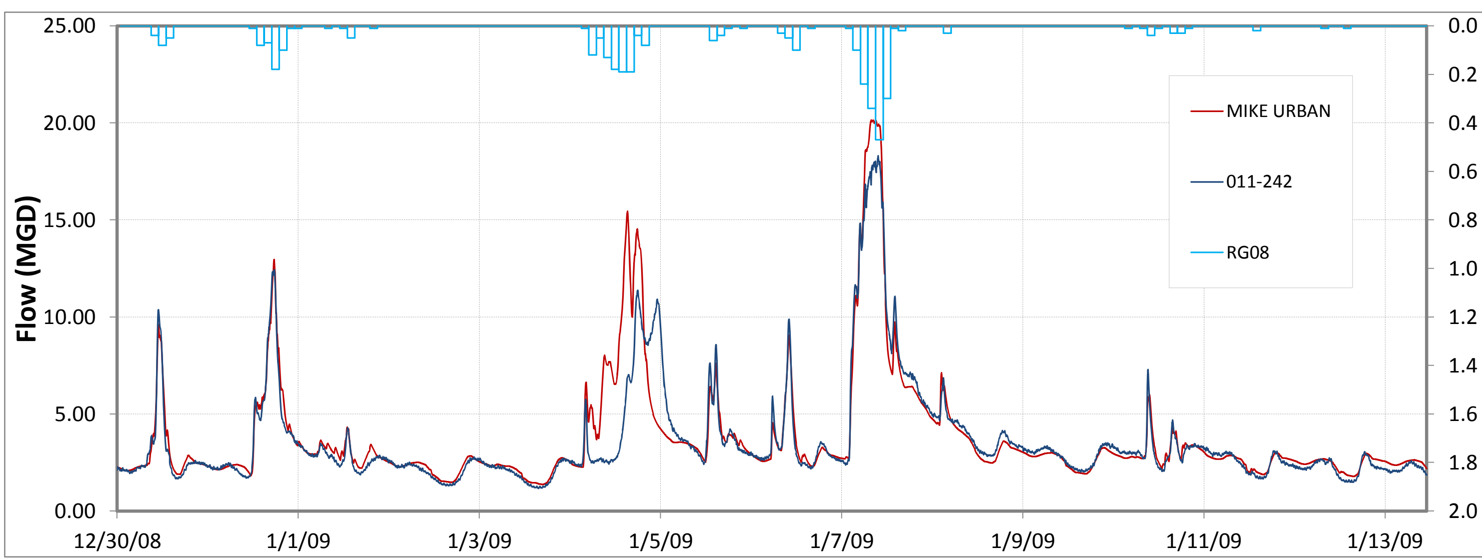


Event 02

10/30/08 to 11/13/08

Parameters	
rdii_area	45.6 %
b_a_iflat	7.6 %
b_m_iflat	0.079
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.10
RMSE	0.87
Nash	0.90
Q_{pk} Error	24.08%
V_{tot} Error	12.09%

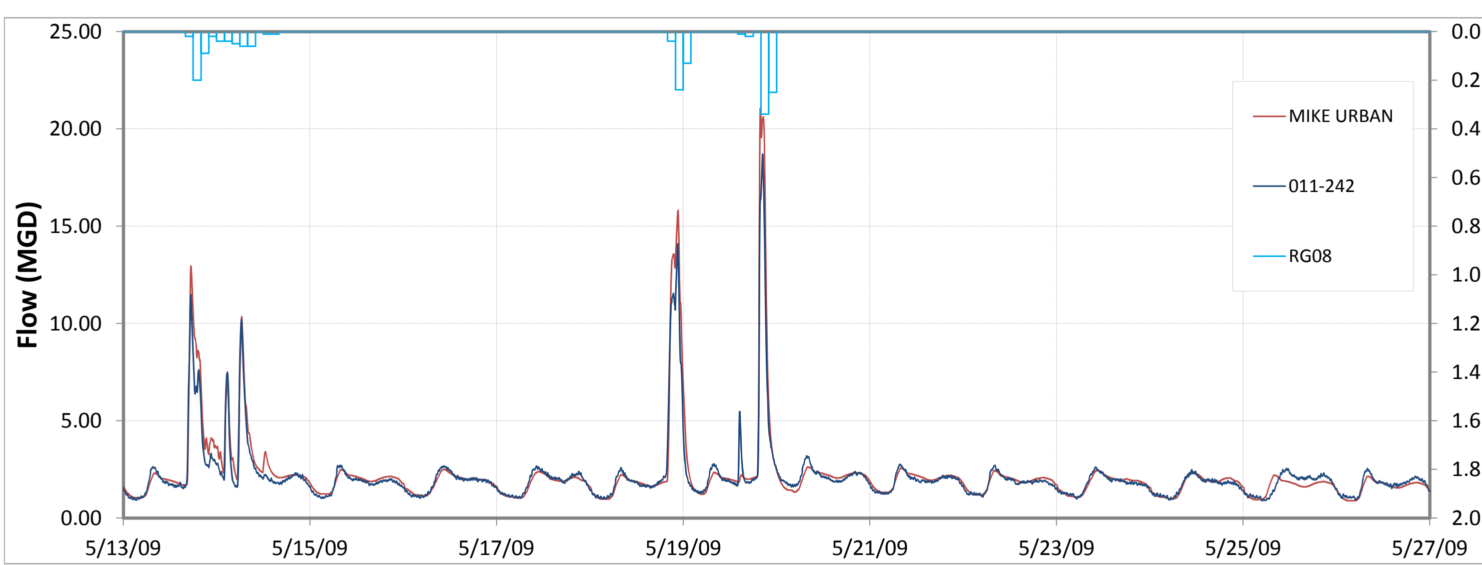


Event 03

12/30/08 to 01/14/09

Parameters	
rdii_area	45.6 %
b_a_iflat	7.6 %
b_m_iflat	0.079
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.04
RMSE	1.17
Nash	0.81
Q_{pk} Error	10.08%
V_{tot} Error	0.61%

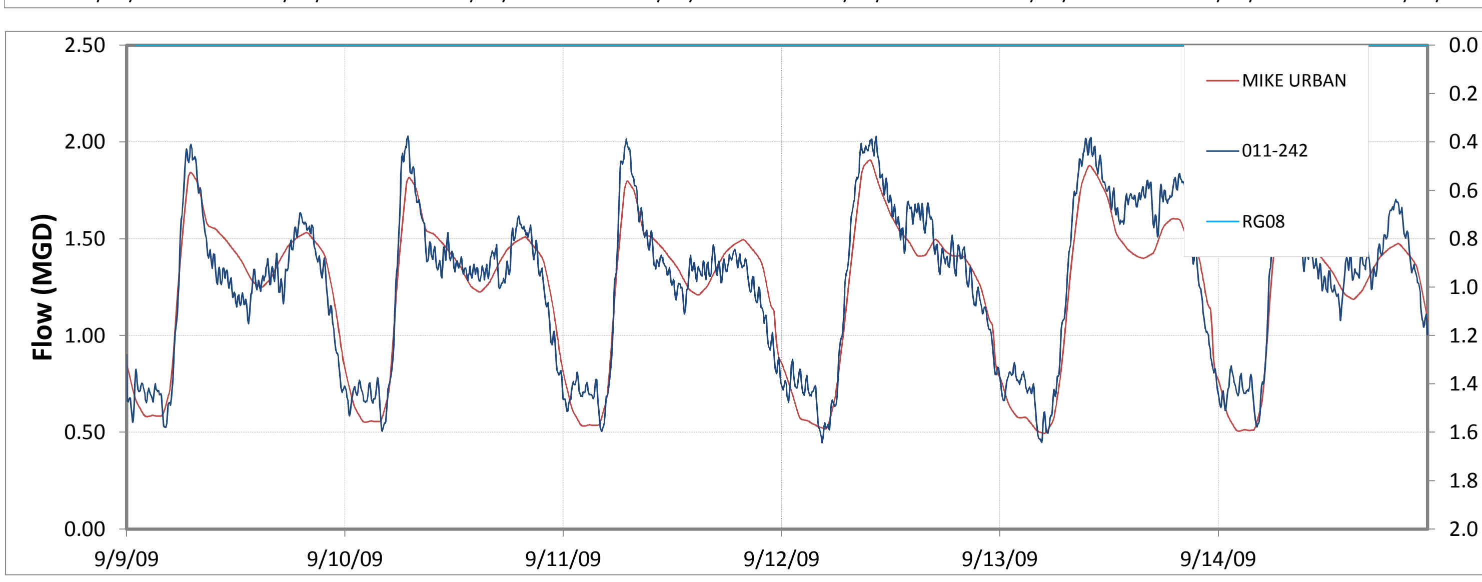


Event 04

05/13/09 to 05/27/09

Parameters	
rdii_area	45.6 %
b_a_iflat	7.6 %
b_m_iflat	0.079
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.04
RMSE	0.48
Nash	0.92
Q_{pk} Error	12.56%
V_{tot} Error	7.18%

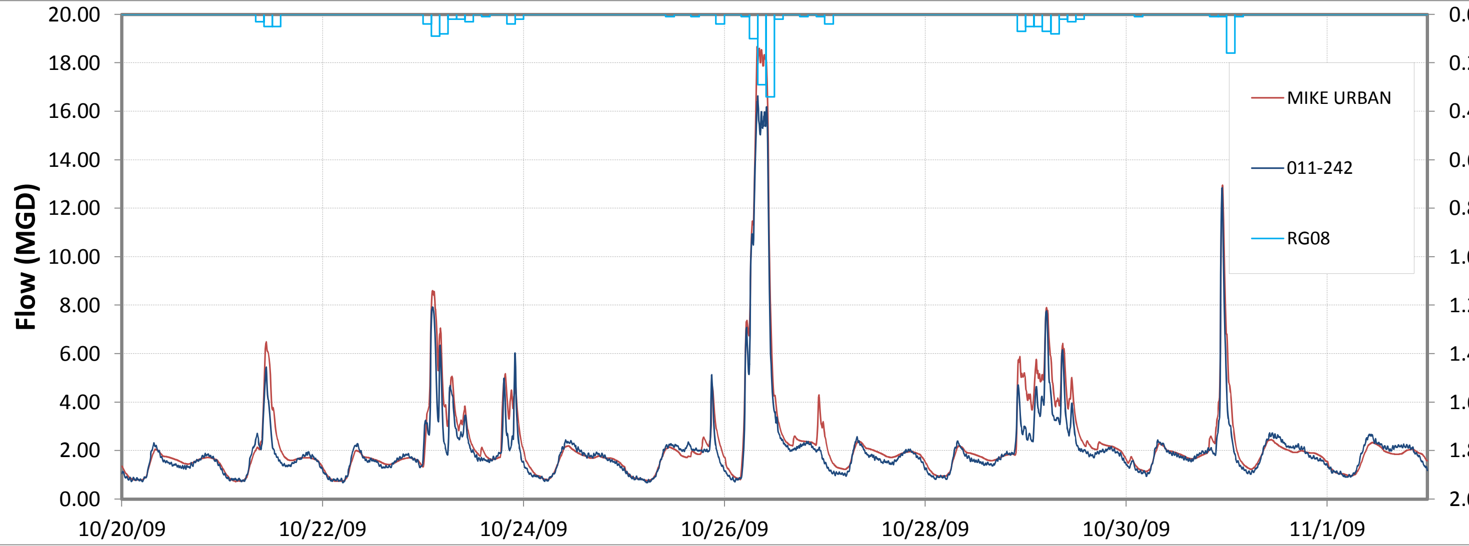


Event 05

09/09/09 to 09/15/09

Parameters	
rdii_area	45.6 %
b_a_iflat	7.6 %
b_m_iflat	0.079
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

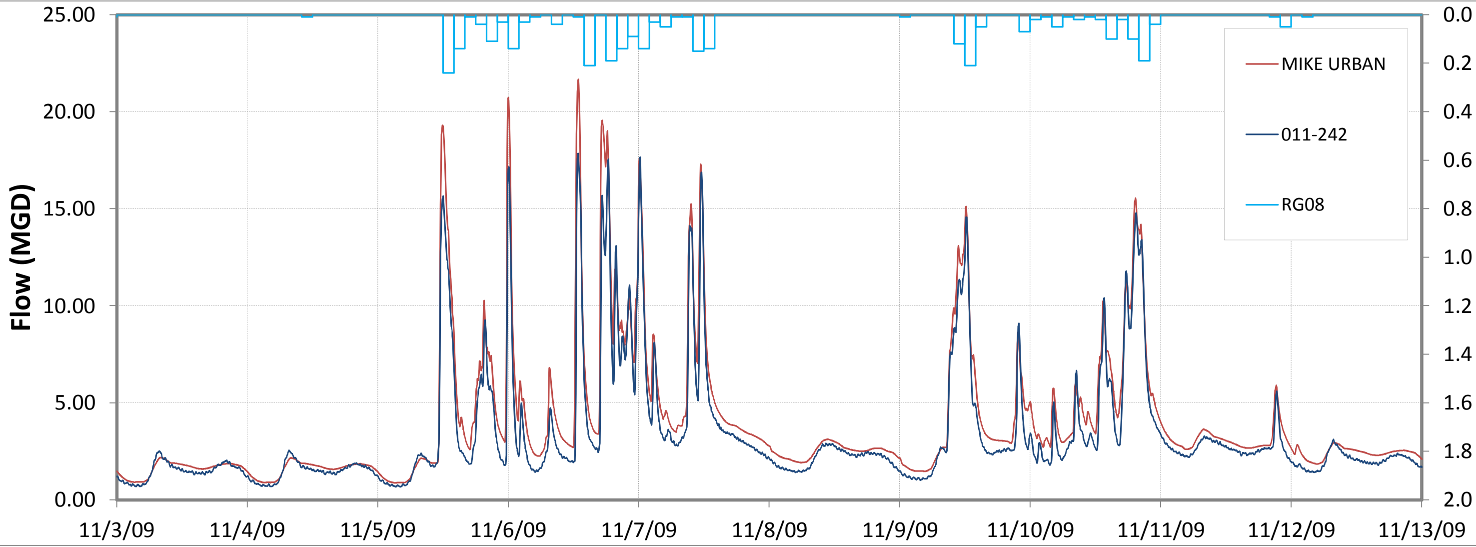
Goodness-of-Fit	
Bias	-0.03
RMSE	0.14
Nash	0.88
Q_{pk} Error	-9.25%
V_{tot} Error	-2.49%



Event 06
10/20/09 to 11/02/09

Parameters	
rdii_area	45.6 %
b_a_iflat	7.6 %
b_m_iflat	0.079
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

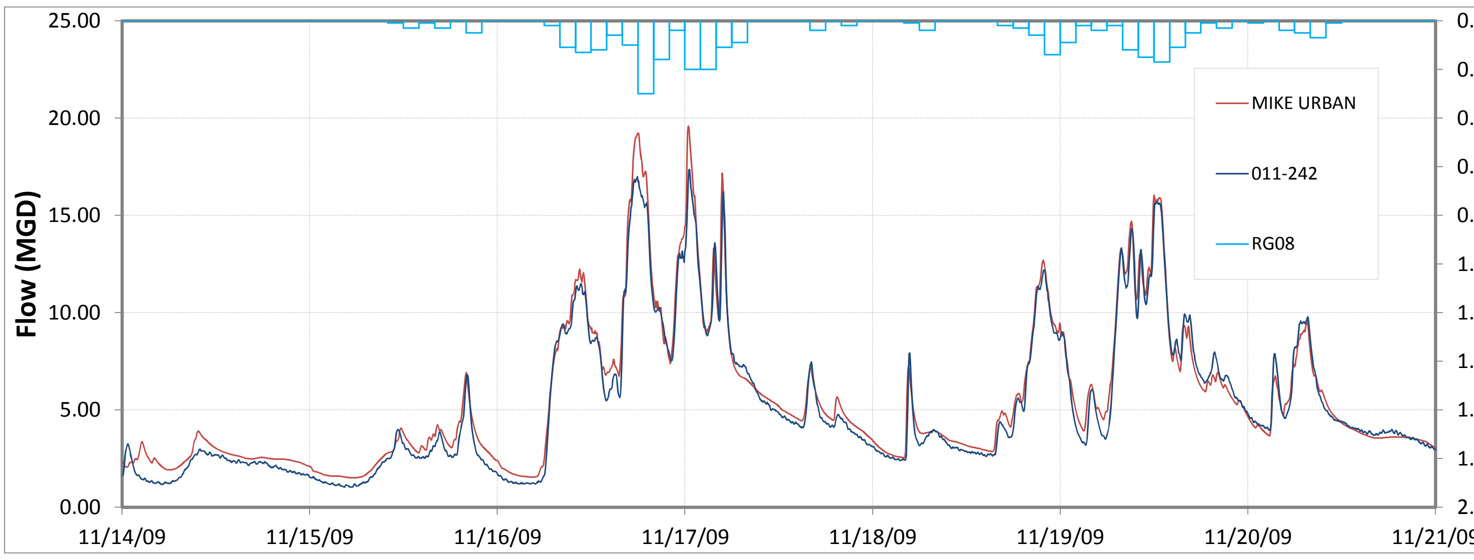
Goodness-of-Fit	
Bias	0.11
RMSE	0.56
Nash	0.91
Q_{pk} Error	12.35%
V_{tot} Error	12.64%



Event 07
11/03/09 to 11/13/09

Parameters	
rdii_area	45.6 %
b_a_iflat	7.6 %
b_m_iflat	0.079
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

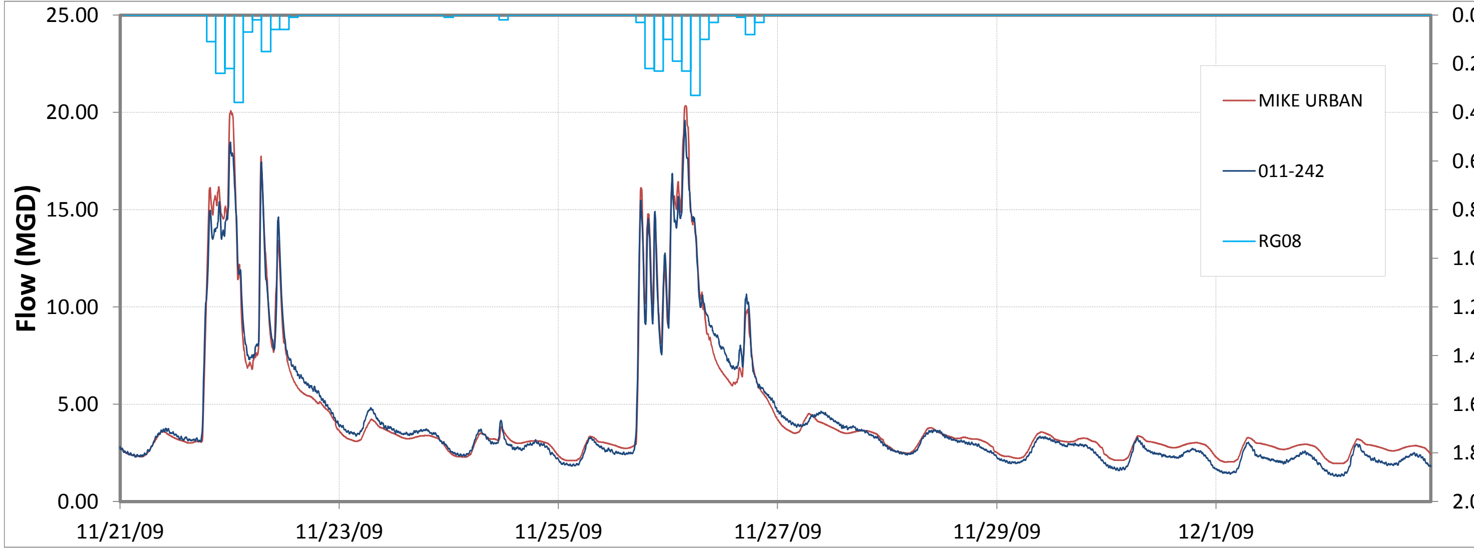
Goodness-of-Fit	
Bias	0.18
RMSE	0.97
Nash	0.90
Q_{pk} Error	21.36%
V_{tot} Error	22.47%



Event 08
11/14/09 to 11/21/09

Parameters	
rdii_area	45.6 %
b_a_iflat	7.6 %
b_m_iflat	0.079
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

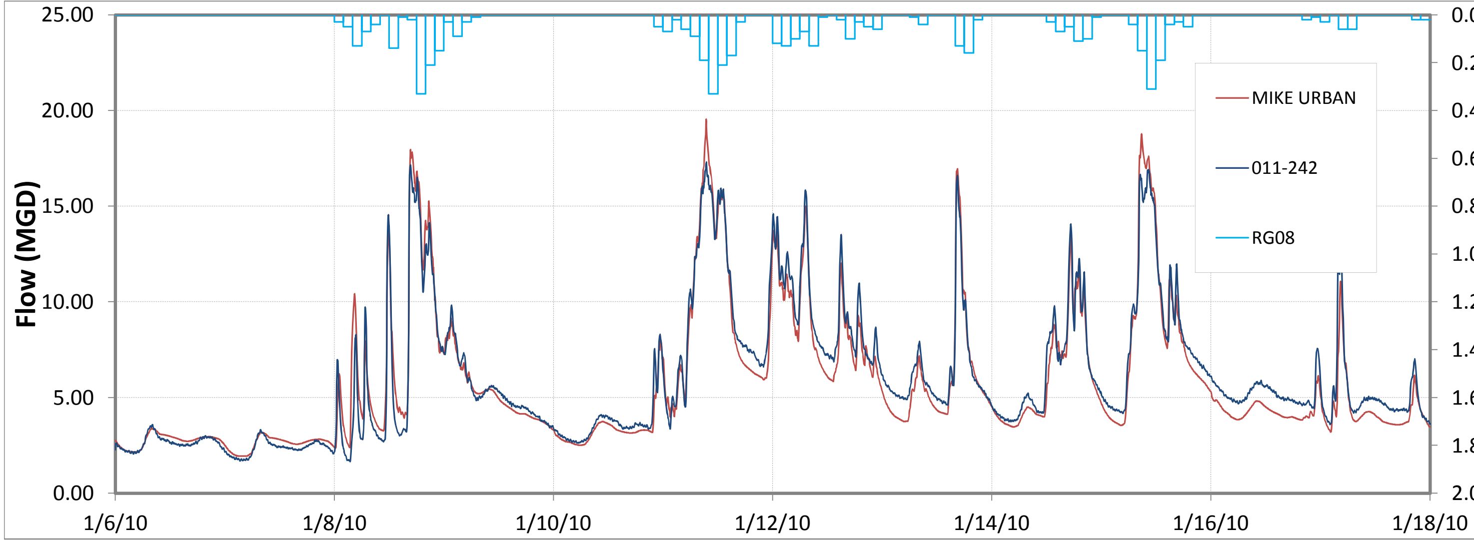
Goodness-of-Fit	
Bias	0.06
RMSE	0.61
Nash	0.97
Q_{pk} Error	12.87%
V_{tot} Error	6.00%



Event 09
11/21/09 to 12/03/09

Parameters	
rdii_area	45.6 %
b_a_iflat	7.6 %
b_m_iflat	0.079
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	0.03
RMSE	0.50
Nash	0.98
Q_{pk} Error	3.89%
V_{tot} Error	-0.75%



Event 10
01/06/10 to 01/18/10

Parameters	
rdii_area	45.6 %
b_a_iflat	7.6 %
b_m_iflat	0.079
umax	1.67 in
lmax	6.14 in
cqof	0.57
ck	7.9 hr
ckif	193.9 hr
ckbf	998.2 hr
i_u	0.39 in
i_l	4.43 in
gw_carea	1.000

Goodness-of-Fit	
Bias	-0.04
RMSE	0.77
Nash	0.95
Q_{pk} Error	12.91%
V_{tot} Error	-4.54%

Appendix D. 11th Ave NW CSO MIKE URBAN Model Development and Calibration Report

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11th Ave NW CSO MIKE URBAN Model Development and Calibration Report

John Conway

January 19, 2018

Table of Contents

Table of Figures and Tables

List of Abbreviations

CSO	Combined Sewer Overflow
CSS	Combined Sewer System
DHI	Danish Hydraulics Institute
DWF	Dry Weather Flow
FRC	Fast Response Component
GIS	Geographic Information System
KC	King County
MGD	Million gallons per day
MH	Manhole
NPDES	National Pollutant Discharge Elimination System
RDII	Rainfall-Dependent Infiltration and Inflow
RMSE	Root-Mean-Square Area
RTC	Real Time Control
SCADA	Supervisory Control And Data Acquisition
SCWQP	Ship Canal Water Quality Project
SPU	Seattle Public Utilities
SRC	Slow Response Component
WTD	Wastewater Treatment Division

1 Introduction

This report documents the development and calibration of hydrologic and hydraulic models for the Combined Sewer System (CSS) for East Ballard in the city of Seattle and their connection to King County's Ballard Trunk. East Ballard overflows discharge through the NPDES permit location WA0029181 (location is at 11th Ave NW Weir). 11th Ave NW Weir has an average of 14 combined sewer overflow (CSO) events per year over the past 15 years. The intent of this model is twofold: 1) to contribute to the development of a comprehensive West Point conveyance model, and 2) to contribute to the development of an Integrated Ship Canal Tunnel Model with the King County North Interceptor model. It is anticipated that the resulting Integrated Tunnel Model and will be used to inform the design and operation of the joint Ship Canal Water Quality Project.

1.1 Model Development Approach

The area upstream of the Ballard Regulator Station (RS) has been divided into sub-basins for ease of calibration using data from a number of temporary flow meters. These sub-basins are used as hydrologic modeling units and each one was calibrated using quality checked data from a unique portable flow meter. King County had previously identified 9 sub-basins within 11th Ave NW Basin and used them in its *Runoff-Transport* model of the West Point System. These sub-basins are used as hydrologic modeling units. Runoff-transport (RT) basins 4 and 9 correspond to 004 & 009, respectively. The sub-basins were imported into the municipal wastewater modeling software MIKE URBAN, developed and distributed by the Danish Hydraulics Institute (DHI), where they were assigned hydrologic parameters and connected to the network of pipes, manholes, and control structures in the area. The appropriate flow meter data was then used to calibrate the hydrologic parameters for each sub-basin, a process automated using the calibration program PEST. Section 4 will describe the hydrologic calibration process in greater detail.

1.2 Service Area and Operation

The 11th Ave NW CSO Area encompasses 1,399 acres of Seattle. It is bounded by Fremont Ship Canal to the south, NW 85th St. to the north, approximately 15th Ave. NW to the west, and approximately Phinney Ave N to the east. Sub-basins 004 and 005 include the part of the CSS that is fully combined. The remaining sub-basins are partially separated, meaning that the stormwater is, in some areas, conveyed via a different pipe than is the sanitary sewage.

The area modeled begins at the conclusion of the Carkeek Force Main (approximately NW 90th St and 8th Ave NW). The model represents the 8th Avenue NW Interceptor down through the Ballard Trunk, including the 11th Ave NW CSO. The downstream boundary of the model is just upstream of the Ballard RS. Section 3.1 discusses the sub-basins in greater detail, and Section 5.1 discusses the 11th Ave NW Overflow weir in greater detail.

The North Beach Pump Station and Carkeek Pump Station models are separate MIKE URBAN models, the output of which is used as input into this 11th Ave NW model during long-term simulations.

2 Data Sources

Data was collected from a variety of sources to develop the hydrologic and hydraulic models. Hydrology for the basin was characterized from photography, contour data, evaporation, rainfall, and existing delineations of hydrologic basins (from King County's *Runoff-transport* model). Hydraulics for the collection system were defined based on as-built drawings, GIS sewer coverages, SCADA, and flow meters. Additionally, portions of the SPU modeling report were used as secondary data sources or to verify inferences and approximations.

2.1 Contour Data

GIS overlays of contour data were used to qualitatively estimate the slope of pipe systems within Ballard. Due to the generally uniform slope of the basin, this was generally only used as an order of magnitude verification of the survey data for the pipes and manholes.

2.2 GIS

Point-based shape files of sewer manholes, and line-based files of sewer pipes, were the primary source of the network data used to parameterize the hydraulic model network. Manhole data includes name, diameter of the pipe intersected, and elevations of the rim and of each inlet and outlet pipe. Pipe data include length, diameter, upstream and downstream invert elevations, and material. Polygon-based shapefiles of KC-WTD CSO basins provided the foundation for the basins used in the hydrologic model, and ArcMap was used to create sub-basins based on flow directions as well as to calculate the area of each sub-basin.

2.3 Evaporation

A long-term evaporation record was downloaded from the AgWeatherNet Washington State University Puyallup site. This record was averaged by month into an average evaporation year, which was repeated for the duration of the calibration and long-term period of simulation. Both this data source and methodology are commonly used for continuous hydrologic modeling in the Puget Sound area.

2.4 Rainfall

Processed rainfall records from 1978 to 2015 were provided by SPU at 17 rain gauges located within the City of Seattle. These records consist of a continuous one-minute time series of rainfall depth, adjusted to Daylight Saving Time. Rain gauge RG07 is located to the north of the basin, and provided the rainfall record for sub-basin 004 in the hydrologic model. Sub-basins 005, 006, 007, 008, 009, 014, and 015 were closest to RG09, located on the eastern boundary of the basin, and so RG09 was used for the hydrologic calibration of these basins. The locations of rain gauges relative to Ballard are shown in Figure 2-1 below.

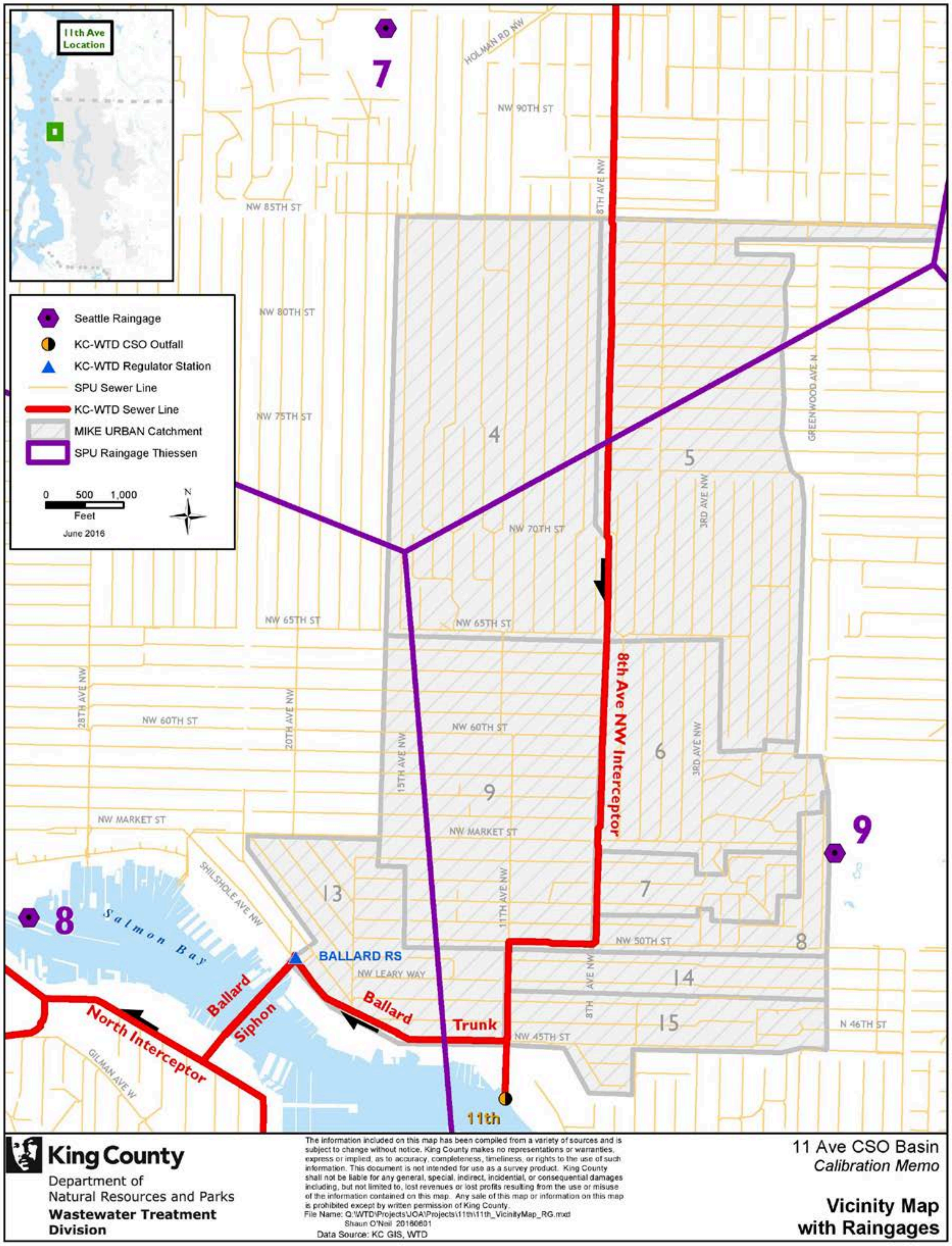


Figure 2-1: Rain gauges and Thiessen polygons in 11th Ave NW CSO Area

2.5 As-Built and Design Engineering Drawings

Archived record drawings of sewer pipes and facilities were available from both WTD and SPU. As-Built and design drawings were used to provide modeling detail for facilities and pipes and to address discrepancies and missing data from GIS shape files.

2.6 SCADA

WTD maintains a historical record of SCADA (Supervisory Control and Data Acquisition) continuously collected from WTD offsite facilities. SCADA data useful for model calibration includes calculated and measured flows, water levels, gate positions, and operational states of pumps and flow regulating structures. SCADA data was used to define a time series for the downstream boundary condition using the trunk elevation at the inflow to the Ballard RS from July 2005 – April 2017. In addition, SCADA was also used for the inflow coming from the Carkeek Pump Station in the Eighth Ave NW Interceptor in the same time period (07/2005 – 04/2017).

2.7 UNSTDY and Runoff Transport Model Runs

King County has previously modeled this basin using runoff transport (RT) and UNSTDY modeling systems. These models have been used as a reference point for links and nodes in this basin, as well as boundary conditions for the Ballard Regulator and Carkeek Pump Station for the periods needed before 2005. The output for the UNSTDY model was used as a downstream boundary condition at the Ballard Regulator, and the Runoff Transport model output was used for the Carkeek Pump Station Effluent flow.

2.8 Flow Monitoring

WTD and SPU perform conveyance system monitoring with an array of permanent and portable flow meters. These are generally installed in manholes, and have collected upstream depth and velocities for at least one wet season, from which flow rates can be calculated. Data is typically sampled at either 5 or 15 minute intervals. The data are used for the hydrology model calibration and for the verification of the hydraulic model.

Table 2-A below shows the meters and periods used for the hydrologic calibration, while Figure 2-2 on the following page shows their locations within the basin.

Table 2-A: Flow meter data quality and locations

Flow Meter	Sub-Basin	Data Quality	Start Date	End Date	Agency	Location
012-005	4	Good	12/3/2015	3/6/2016	KC	NW 64th St & 11th Ave NW
012-413	5	Good	12/19/2015	3/7/2016	KC	NW 62nd St & 7th Ave NW
012-373A	6_7	Good	9/24/2009	3/29/2010	SPU	NW 51st St & 8th Ave NW
012-165	8_14	Good	9/1/2009	4/1/2010	KC	NW 45th St & 11th Ave NW
012-127A	9	Good	9/19/2009	3/22/2010	SPU	NW 51st St & 11th Ave NW
012-162	15	Good	12/17/2015	3/7/2016	KC	NW 45th St & 11th Ave NW

For this project, 3 King County portable flow meters were installed in the basin in December 2015. Sub-basin 4 and 5 are both fully combined systems, and it was determined that flow meters in these sub-basins would provide an opportunity for us to more accurately reflect the hydrologic conditions of these basins. Sub-basin 15 roughly matched the geographic parameters of length and slope to that of sub-basins 8 and 14, so 1 flow meter was installed in sub-basin 15 and the parameters found for that metered basin were initially used for sub-basins 8 and 14. Flow Meters 012-005, 012-413 & 012-162 were all chosen based on flow characteristics of the pipe, slope, and their downstream location in each corresponding sub-basins.

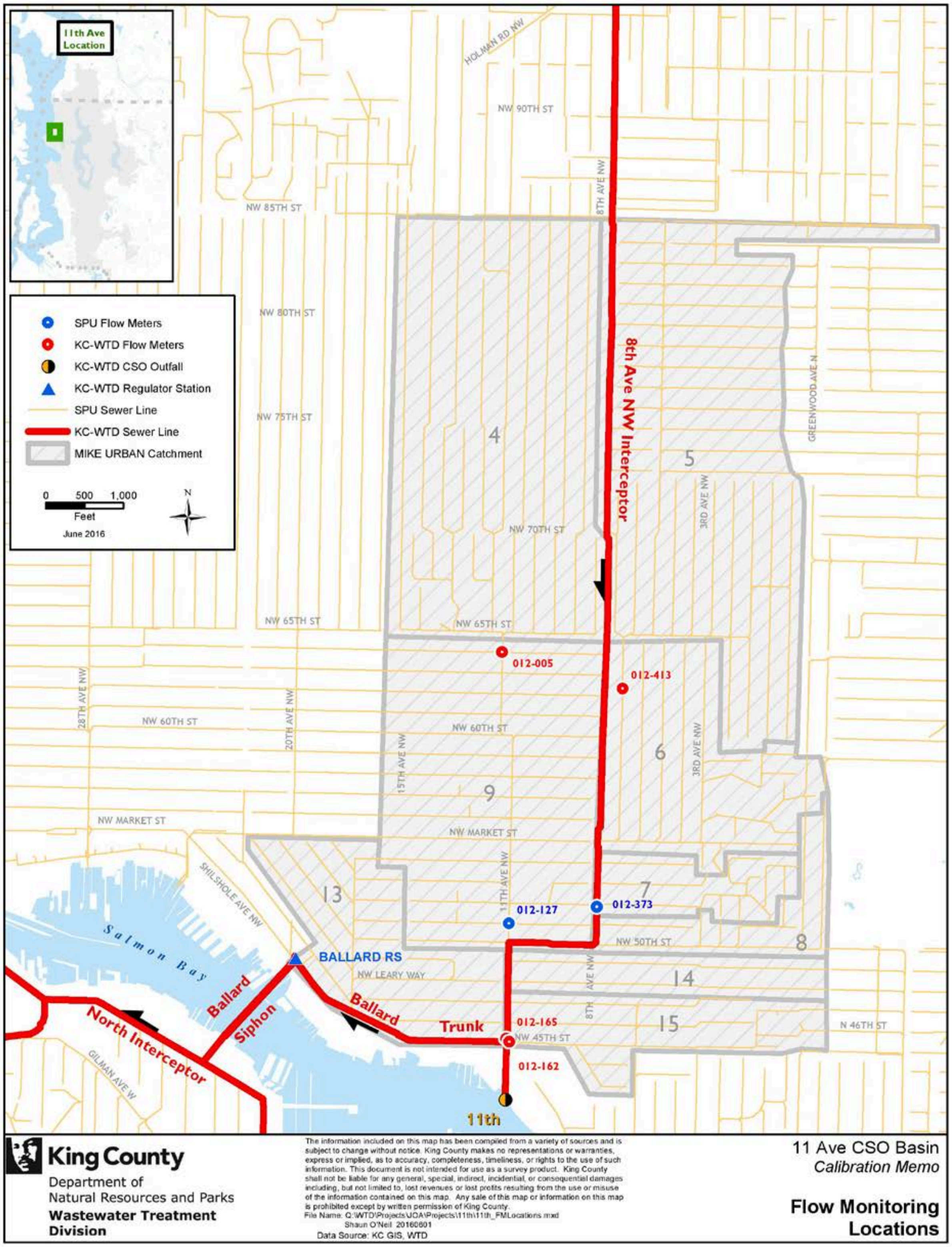


Figure 2-2: Locations of flow meters in the 11th Ave NW System

3 Model Development

The Danish Hydraulic Institute (DHI) MIKE URBAN model was selected to perform hydrologic and hydraulic modeling of the Ballard CSS. The model consists of a hydrologic component and hydraulic component.

Model development consisted first of compiling the different elements of the model (basins, manholes, pipes, and control structures, which, in the case of this particular basin, included the two overflow weirs), connecting them, and assigning them properties consistent with their physical attributes. Second, boundary conditions including rainfall, evapotranspiration, dry weather flows (DWF), and Salmon Bay and Ballard RS water levels were used to define the inputs for the network. Third, the hydraulics of the conveyance were adjusted, including the physical characteristics of the weir and the head losses at nodes, as the default parameters in MIKE URBAN tend to compute losses that are higher than monitoring data would suggest are realistic. In this particular model, results of the hydraulics were not reconciling with the hydrographs, so more flow data was obtained, finding that the originally anticipated hydraulics of the weir were underestimating overflows. The model was adjusted using new flow data; the new values for flow were used to verify the hydrologic and hydraulic model.

Figure 3-1 below highlights the modeled conveyance piping, the control structures, the overflow points, and the connection to the Ballard Regulator Station.

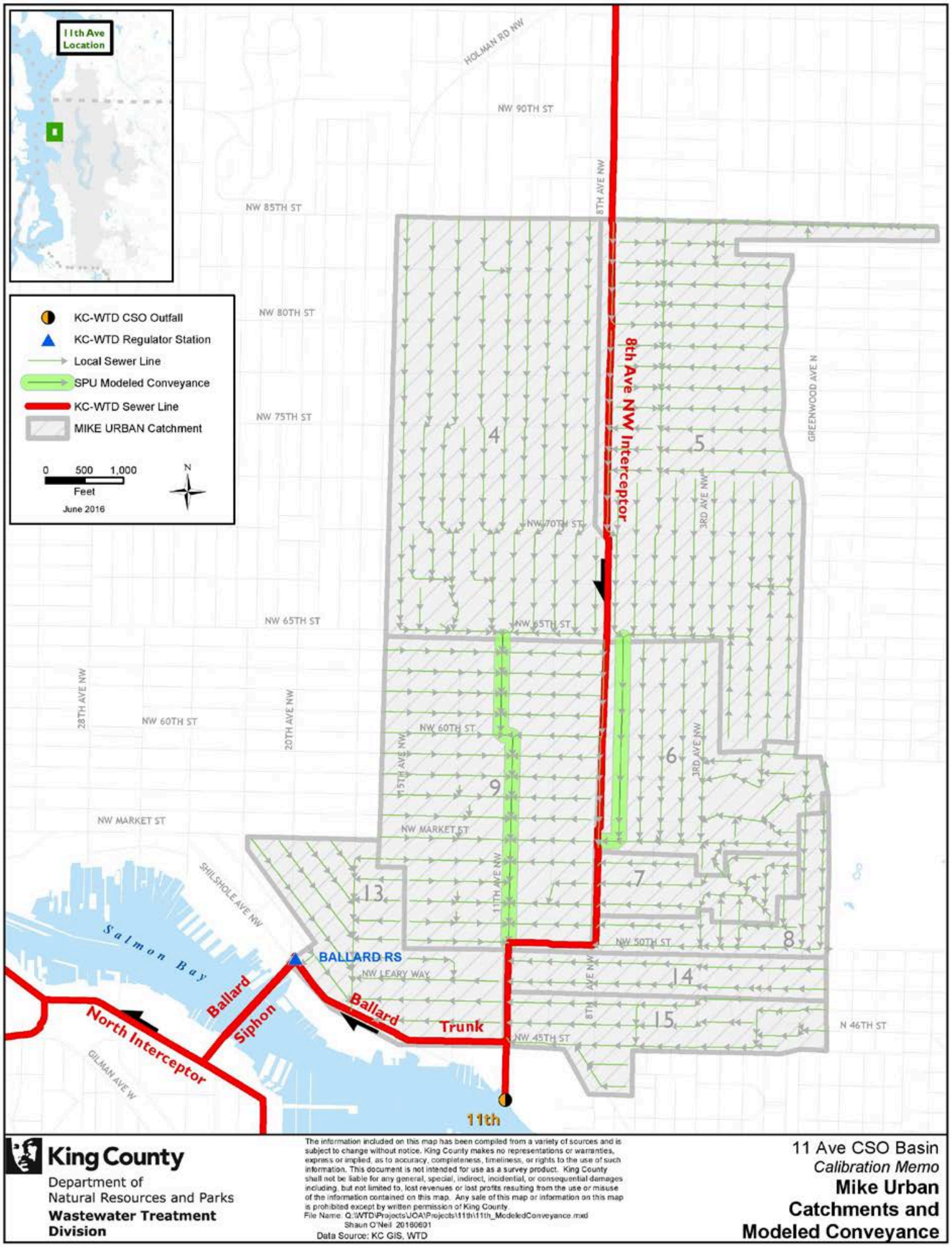


Figure 3-1: Modeled catchments of 11th Ave NW & Ballard Trunk

3.1 Hydrologic Model

The hydrologic component (MOUSE RDII [Rainfall Dependent Infiltration and Inflow] and MOUSE Runoff Computation) characterizes the basin response to rainfall in terms of a hydrograph at a tributary location. The hydrograph consists of both surface runoff and RDII. Surface runoff routing is calculated using the Kinematic Wave model (Model B), and accounts for runoff produced from impervious areas of the basin. It's often referred to as the fast response component (FRC). The RDII module accounts for overland flow, interflow, and groundwater processes related to pervious areas of the basin. It's often referred to as the slow response component (SRC). RDII considers evapotranspiration, soil moisture, and groundwater table depth in its calculations.

The process of delineating each sub-basin began by using the previously-defined basins for WTD's *Runoff-Transport* model of the Ballard area, which was based on flow directions of pipes throughout the network. The drainage area of each basin, as well as the relevant flow meter, is given in Table 3-A below.

3.1.1 Summary of Sub-Basins

Table 3-A below summarizes the sub-basins calibrated along with their respective areas, associated rain gauge, and flow meter used for calibrating the sub-basin.

Table 3-A: List of Calibrated Model Basins

Sub-Basin	Basin Area (acres)	Rain Gauge	Flow Meter
4	334.85	RG07	012-005 (KC)
5	344.38	RG09	012-413 (KC)
6	136.37	RG09	012-373 (SPU)
7	46.32	RG09	012-373 (SPU)
8	46.886	RG09	11THAVENW (KC)
9	248.68	RG09	012-127A
13	114.07	RG08	--
14	51.54	RG09	11THAVENW (KC)
15	76.1	RG09	012-162 (KC)

3.1.1.1 Sub-basin 004

Sub-basin 004 was calibrated using a new portable flow meter, installed in December 2015. This basin was isolated with its own flow meter to represent the hydrologic conditions in the sub-basin, as nearly all of the area comprising this region is served by a fully combined sewer system.

3.1.1.2 Sub-basin 005

Sub-basin 005, similar to sub-basin 004, had a portable flow meter installed in December 2015. The downstream flow meter incorporated areas that were both combined and partially separated, so in order to more accurately reflect the hydrologic conditions of the fully combined area, this flow meter was installed.

3.1.1.3 Sub-basin 006_007

Sub-basins 006 and 007, as delineated by King County in their runoff transport models, have been combined to make 006_007 for this calibration. This was due to the similarities as partially separated basins with similar slopes and geometric conditions. The flow meter for this sub-basin measured flow that includes flows from Carkeek PS and sub-basin 005. The flow meter is located on a manhole in the 8th Ave NW Interceptor.

3.1.1.4 Sub-basin 008_014

Sub basin 008 & 014, while separate in Runoff transport, were considered conjoined during calibrations. They were kept separate in Mike Urban for ease of differentiation, if flow meters were to be utilized in either sub-basin in the future. These basins made up a small percentage of the overall contributing area, and were not directly monitored. These basins were "calibrated" in such a way as to improve the model match at the 11th Ave NW overflow structure. It is because of this that the RDII area and impervious area of these basins appear so large.

3.1.1.5 Sub-basin 009

Sub-basin 009 is located downstream of basin 004, and was calibrated using an SPU flow meter with flow data from 2009-2010. The flows in this system feed into the Ballard Trunk, and comprise the. This basin was calibrated after sub-basin 004 was calibrated to 2015-2016 data. The calibrated parameters from sub-basin 004 were input to 009 as an upstream boundary condition when calibrating sub-basin 009.

3.1.1.6 Sub-basin 013

Sub-basin 013 is downstream of any usable flow meters in this basin, and therefore was not directly calibrated. Ultimately, the parameters used in this basin were determined using the following methodology: impervious area was determined using a high-end estimate of the percent impervious connected, from a King County GIS study performed in 2010. RDII area was determined using high-end estimates for similar sub-basins in the 11th Ave NW System, such as sub-basin 005. Because data downstream of the 11th Ave NW weir structure showed more RDII slow response, the other RDII calibration parameters selected were intended to support more slow response, due to the desire to more accurately match those slow response flows headed towards the Ballard Regulator. These were picked from previous model runs in other basins outside 11th Ave NW, but chosen to accommodate more flow in the system for the week following a major storm event.

3.1.1.7 Sub-basin 015

Sub-basin 015 had a flow meter installed early in the 11th Ave NW model development. This flow meter was used to gain a ballpark sense of the parameters for sub basins 008 & 014, until the realization that more flow was needed at 11th Ave NW and those basins were used to incorporate those flows. Sub-basin 015 had a KC meter installed just upstream of the entry point to the Ballard Trunk, which is a few feet upstream of the 11th Ave NW weir structure.

3.1.2 Summary of Hydrologic Parameters

The FRC requires length and slope parameters for each basin. The length was set to a standardized 300 ft, which seems reasonable given the maximum reach until the water will reach the open channeled pipe in the system. Slopes were determined using contour information in GIS across the length of the basin that flow generally travels, and divided by the approximate length of the basin. Slopes were set to a value in per thousand (‰), not percent (%), due to the nomenclature in MIKE UBRAN. A slope of 5.140% in basin 004, for example, led to a input value of 51.40 ‰. This appeared to lead to larger Manning's roughness as calibrated by MIKE UBRAN. This seemed acceptable due to the fact that the Manning roughness obtained during the hydrologic calibration will adjust as a reflection of any deviation in the slope and length in each basin from the constant values set in the model. The parameters chosen to be calibrated are provided in Table 3-B, and the rest of the hydrologic parameters are set to MIKE URBAN default values.

Table 3-B: Parameters calibrated for the hydrologic model

Parameter Name	Model ID	Description	Units
Impervious Area – Flat	B_A_IFLAT	Fraction of basin area that is impervious and connected to the CSS	%
Impervious Manning Number - Flat	B_M_IFLAT	Defines the roughness of the impervious basin area, used in the hydraulic routing of the runoff (Manning's equation)	N/A
RDII Area	RDII_AREA	Fraction of basin that contributes RDII to the CSS	%
Surface Storage	UMAX	Defines the maximal water contents in surface storage	In
Root Zone Storage	LMAX	Defines the maximal water contents in root zone storage	In
Overland Coefficient	CQOF	Determines the extent to which excess rainfall (after surface storage is retained) runs off as overland flow	N/A
Time Constant	CK	Determines how fast the flow responds to rainfall. Also has some effect on the routing of interflow.	Hr
Time Constant Interflow	CKIF	Together with Umax determines the amount of interflow	Hr
Time Constant Baseflow	CKBF	Determines the hydrograph recession during dry periods	Hr

In addition to these parameters *I_U* and *I_L* (the initial abstractions for surface storage and root zone storage) were fixed at the initial value of *UMAX* and 75% of the initial value of *LMAX*, respectively. The variable *GW_CAREA*, the proportion of the groundwater area to the basin area, was also originally used during PEST calibration, but was later fixed at 1.0 as recommended by the reviewers of the model.

3.2 Hydraulic Model

The hydraulic component (MOUSE HD Pipe Flow Computation) further characterizes the hydrologic response to rainfall in terms of flow and water levels within the CSS. MOUSE HD Pipe Flow uses a Dynamic Wave model to route flow within a network of nodes and links. Nodes represent manholes or outlets; links represent pipes, storage tanks and control devices such as pumps, valves, weirs and regulated gates. The hydraulic network used to represent the Ballard trunk is highlighted in green in Figure 3-1. Section 3.6 describes the control structures in further detail. Most parts of the hydraulic model were not calibrated per se; instead, existing dimensions, facility operation manuals, engineering drawings, and monitoring data were used to match the model's hydraulic parameters to physical observations, and adjustments were made where necessary to ensure model stability and realistic results. The only exception is with the overflow structures, where a variety of approaches were used in an attempt to match simulated overflow data to observed overflow data. This calibration is discussed in detail in Section 5.

3.3 Boundary Conditions

The boundary conditions, or loading inputs, for the hydrology are rainfall and evapotranspiration. These have been described in Section 2. The boundary conditions for the hydraulic model are upstream inflow to the CSS, dry weather flow (DWF) and downstream water levels.

3.3.1 Upstream Inflow

Inflow into the hydraulic model network is generally represented as a time series of metered flows or modeled results. The latter is a common alternative to expanding the network model to upstream basins and networks. The main example of upstream inflow in this model was the Carkeek Pump Station, location upstream of the Eighth Ave Interceptor. The flows from this location were input as a time series. The source of these flows was typically SCADA (from 2005-2017), but UNSTDY model results were also used to expand to long term simulations back to 1978.

3.3.2 Dry Weather Flows

The DWF is the combination of groundwater infiltration and sanitary flow regularly entering the collection system regardless of precipitation. Meter data during dry weather periods were used to create a set of dry weather diurnal curves for all basins. Three diurnal curves were created for each basin to reflect average observed flows during weekdays, Saturdays, and Sundays. The 5-minute flow meter data was averaged into hourly flow rates to get the 3 DWF patterns for each sub-basin. When available, flow meter data from the dry weather period from 7/14/09 to 8/8/09 was used to determine representative DWF averages for BALL_9 and BALL_6_7. For BALL_4, BALL_5, and BALL_15, the period used was from 8/8/2016 – 8/30/2016. For BALL_13, the diurnal was set to the BALL_08_14 pattern, and scaled to its appropriate magnitude using a ratio of areas.

3.3.3 Downstream Water Levels

At each of the outlets of the hydraulic model (the overflow outfall on 11th Ave NW and the connection to the Ballard Regulating Station), a water level was used as a downstream boundary condition. For each overflow outfall, the ship canal water level was provided by the Army Corp of Engineers and SCADA data. This value is a daily value, and adjusts as the ship canal gets lowered in the fall and raised in the spring.

At the outlet of the model into the Ballard RS, Ballard Trunk SCADA data was used to develop a time series of the water level in the inlet trunk. Additional modeling using UNSTDY was necessary to extend the time series and cover the entirety of the long term simulation, back to 1978.

3.4 Pumps and Control Structures

This model includes the one main control structure: an overflow weir at 11th Ave NW. The Ballard RS was not modeled as part of this basin, and instead its Ballard Trunk level was the downstream boundary condition.

3.4.1 11th Ave NW Overflow Weir

On the Ballard trunk at the intersection of NW 45th St and 11th Ave NW, there is a 29.43' long overflow weir that controls flows coming in from the surrounding sub basins and the Carkeek Pump station. The overflow weir height is set to approximately 36.6" above the pipe invert, which equates to 118.16 King County Metro Datum (KMD). During the initial calibration period (2006 – 2015), the reported information indicated that this weir experienced an average of 16 overflow events per year and an average of 11.4 million gallons (MG) discharged per year. In the development of this model, it was discovered that these values appear to underestimate the amount of overflow actually occurring in this structure. The calibration process for this weir is discussed in Section 5.1.

3.5 Spilling Nodes

In MIKE URBAN, surcharging nodes can be set to several different conditions. In King County Models, the standard setting for nodes is set to spilling, which allows for flow in a surcharge manhole to leave the system, and does not have the flow re-enter the system. This is done as an alternative to normal and sealed, which do not replicate the real-life conditions of most manholes in the system.

3.6 Losses in Nodes

Head losses in nodes such as manholes and junctions are computed in Mike Urban. The standard calculation usually overestimates the losses, and thus custom losses were set for the nodes in the model. The custom loss estimates follow the Weighted Inlet Energy method (also referred to as Mean Energy Approach), with a coefficient type of total head loss (Total HLC) applied at the outlet of the node. Five standard cases were used in this model. The loss coefficients used are listed in Table 3-C. Some customized losses using the Weighted Inlet Energy Levels (WIE) approach were also applied.

Table 3-C: Head Loss Coefficients

Type of Node	Loss Coefficient
45 Bend	.1
90 Bend	.2
Channelized	.05
Junction	No head loss
Outlet	1

For a few select nodes, the node type was set to "Junction" to manually force the head loss through the node to be zero.

3.7 Sediment in Pipe

From the point in which the Ballard Trunk transitions from a circular concrete pipe to a brick channel (approximately KC Manhole LU16-05 & SPU ID 012-172), approximately .3' of silt has been found in the pipes downstream. This phenomenon continues until nearly the Ballard Regulator. In an attempt to accurately display the flows and water levels coming into the weir structure, the model incorporated this sediment into its model by trimming the bottom section of the pipe and nodes to .3' higher. In rectangular culverts, this meant raising the invert .3'. In circular pipes (as exist from the 11th Ave NW weir structure down to the Ballard Regulator), this meant changing the cross-sectional geometry of the pipe in MIKE UBRAN using a .crs file. This practice improved the results of the model, and also replicated the real-life conditions more wholly.

3.8 Real-time control (RTC)

RTC was not used due to the lack of any movable control structures in the basins.

4 Hydrologic Calibration

Model calibration is the process of iteratively adjusting model parameters until the results of the model most closely approximate real world observations. This is done by adjusting the hydrologic parameters of the model for each calibrated basin shown in Table A-2. This iterative process has been automated using Model-Independent Parameter Estimation and Uncertainty Analysis (PEST) software. PEST employs a Gauss-Marquardt-Levenberg method of steepest descent to minimize the differences between model results and observations. Sometimes, after the automatic calibration, a manual adjustment of the parameters is carried out to improve the match between model and observations.

Prior to calibration, events are first identified from the monitoring data and assigned appropriate weighting factors. Events with snow are avoided, as it is difficult to model melt rates accurately. Likewise, strong wind events can lead to rainfall measuring errors and should also be avoided. During calibration, the model is run for a sufficient duration to encompass the calibration events. Following calibration, the goodness-of-fit of the results are evaluated using several statistical criterion and metrics. A spin-up period of at least 2 wet seasons prior to the first calibration event is recommended for the hydrologic calibration.

4.1 Events

When possible, suitable events for calibration were manually selected by reviewing the meter time series. Ten flow events of varying magnitude and duration were identified for each sub-basin. Generally, two events were selected as representative DWF periods, and the remaining eight events were the 8 largest storms in the flow meter date range. Some notable deviations from this approach were:

- Sub-basin 009: Event 08 had snow in the second half of the event for multiple days, from 1/11/2010 – 1/15/2010. This period received a 0 weight in PEST.
- Sub-basin 006_007: Event 02 had poor flow data readings on 10/17/2009, which led to a 0 weight in PEST for just over 24 hours.

Given the variability in the time span of metered data for each location (see Table A-1), not all basins were calibrated to the same events. The periods used in the calibration are evident in the calibration plots in section 6.

4.2 PEST Weighting

PEST has the ability to weight individual observations during the calibration. PEST will dedicate more computational effort to matching observations with higher weights, and less effort for lower weights. Observations with zero weight are effectively ignored.

The selection of an appropriate weighting strategy is dependent largely upon the nature of the calibration and PEST's performance without a weighting scheme. For this particular model, the calibration performed adequately without the use of a variable weighting scheme. All valid values were assigned a weight of 1, and missing or clearly inaccurate data were assigned a weight of 0 to eliminate their contribution from the calibration error measurement and decision process. The only portion of the time series that were consistently given a weight of 0 was the period from January 4th, 2009 at 13:00 to January 5th, 2009 at 21:00. The flow data in this timeframe includes snowfall and snowmelt, which resulted in a mismatch in the timing of the response flow and as a result was unfit for calibration purposes.

4.3 Manual Calibration

After the PEST calibration, an analysis of the resulting RDII flow components showed which of those were under- or over-estimated. Adjustment of the parameters based on this information improved the model fit to data for most basins. In general, manual calibration was used to adjust the hydrologic parameter *B_A_Iflat* and was used to attain a more desirable fit for the peak flow values in cases where the model consistently under- or over-estimated the peak flow values. This occurred to a more extreme level in basins 008 & 014.

4.4 Evaluation of Results

The statistical criteria used to evaluate the goodness-of-fit between measured observations and calibrated model predictions for this modeling effort are the standardized Bias, Nash-Sutcliffe model efficiency coefficient, the Root-Mean-Square Error (RMSE), the Relative Peak Flow Difference and Relative Total Volume Difference.

4.4.1 Bias

Bias indicates a general shift of the models, and ranges from positive to negative infinity, where 0.0 indicates a perfect fit. Positive and negative bias indicates model overestimation and underestimation, respectively. Bias is expressed as:

$$B_s = \frac{\sum_{i=1}^{m}\left(Q_i^{mdl} - Q_i^{obs}\right)}{m * \overline{Q^{obs}}}$$

where m is the number of values in the calibration event, Q_i^{mdl} is a model value, Q_i^{obs} is an observed value, and $\overline{Q^{obs}}$ is the average observed value.

4.4.2 Nash-Sutcliffe Coefficient

The Nash coefficient is one less the ratio of the sum of the squared differences between modeled and observed values and the sum of the squared differences between the observed and mean observed values. Nash values can range from 1.0 to negative infinity, where 1.0 indicates a perfect fit. In practice, the Nash coefficient served as the primary goodness-of-fit indicator. Values of 0.9 and greater indicated excellent agreement between the observations and the model. The Nash coefficient is expressed as:

$$Nash = 1 - \frac{\sum_{i=1}^{m}(Q_i^{mdl} - Q_i^{obs})^2}{\sum_{i=1}^{m}(Q_i^{obs} - \overline{Q}_i^{obs})^2}$$

where m is the number of values in the calibration event, Q_i^{mdl} is a model value, Q_i^{obs} is an observed value, and $\overline{Q}_i^{obs}$ is the average observed value in the event.

4.4.3 Root-Mean-Square Error

RMSE is the square root of the average squared difference between the observed and model values. RMSE values can range from 0.0 to positive infinity, where 0.0 indicates a perfect fit. RMSE is expressed as:

$$RMSE = \sqrt{\frac{1}{m}\sum_{i=1}^{m}(Q_i^{mdl} - Q_i^{obs})^2}$$

where m is the number of values in the calibration event, Q_i^{mdl} is a model value, and Q_i^{obs} is an observed value.

4.4.4 Relative Peak Flow Difference and Relative Total Volume Difference

Two additional metrics further quantified the goodness-of-fit. These compared the relative difference of model peak from observed peak, and total model volume from total observed volume. Values of 0.0 for both peak flow and total volume indicate perfect agreement. Positive or negative values indicate model overestimation or underestimation, respectively. The relative peak flow and relative total volume are expressed as:

$$Q_{pk\_diff} = \frac{Q_{\max}^{mdl} - Q_{\max}^{obs}}{Q_{\max}^{obs}} \; ; \; V_{diff} = \frac{V_{tot}^{mdl} - V_{tot}^{obs}}{V_{tot}^{obs}}$$

where $Q_{\max}^{mdl}$ is the maximum model value and $Q_{\max}^{obs}$ is the maximum observed value, V_{tot}^{mdl} is the total model volume, and V_{tot}^{obs} is the total observed volume.

4.5 Hydrologic Parameters

The hydrologic parameters calibrated in the model were discussed in Table 3-B. Calibrated values for these parameters are tabulated by basin in Section 6.

5 Hydraulic Calibration

In order to accurately model the frequency and volume of CSO events, the King County overflow structure at 11th Ave NW required calibration.

5.1 11th Ave NW Overflow Weir

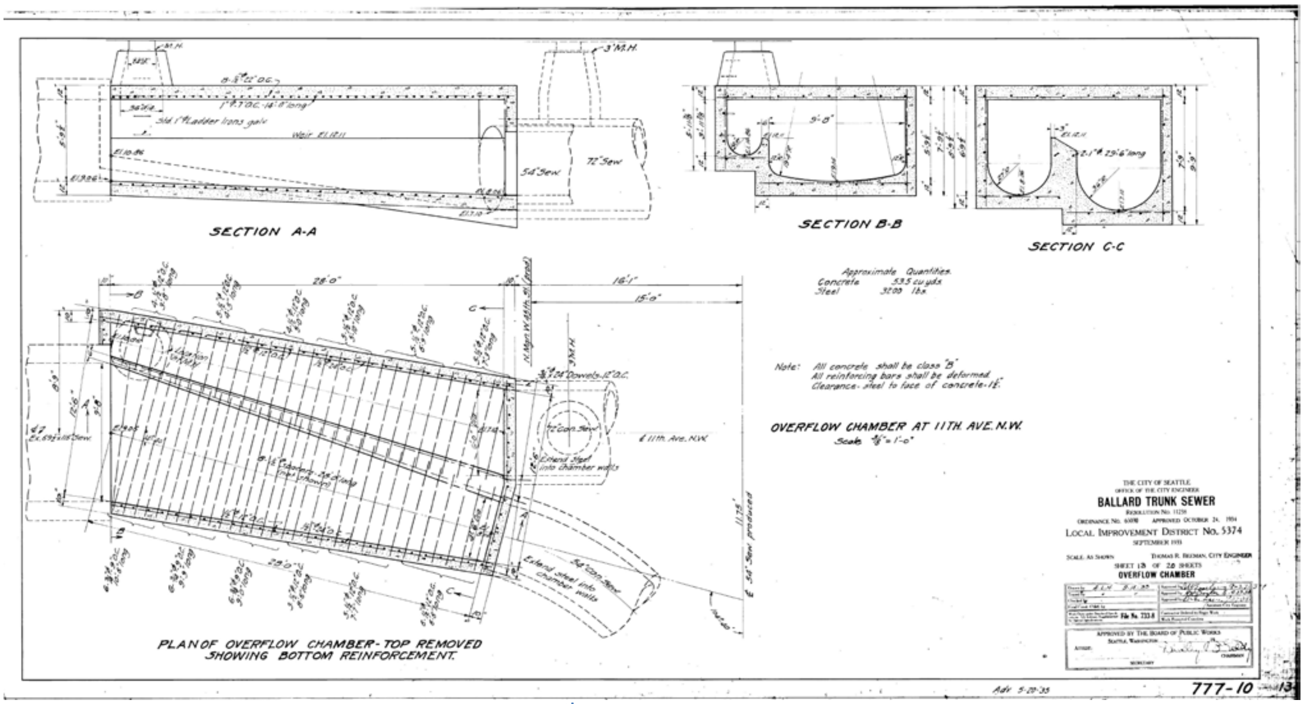


Figure 5-1: Engineering drawing for 11th Ave NW Overflow Weir.

The overflow structure at 11th Ave NW is a transverse weir, but behaves similar to a perpendicular broad-crested weir. Since the main cause of overflow is backwatering that occurs upstream of the Ballard RS, causing the water level to rise in the weir chamber in a sub critical state. The inlet pipe is a brick culvert, with approximate dimensions of 66.5” tall and 116” wide. In the weir structure, the in-line conveyance transitions to a 54” circular concrete pipe. The overflow pipe is a 72” circular concrete pipe, which has a reduced section approximately 230’ downstream.

The results of the hydraulics of the 11th Ave NW model were compared to King County’s reported overflow frequencies and volumes reported in the annual CSO Consent Decree Reports. These reported values were calculated using SCADA level data, a weir height of 36.6”, a weir length of 29.43 feet, and a weir coefficient of 3. Taking the measured water level just upstream of the weir, King County has been reported overflows using that water level and the weir equation to come up with a value for flow over the weir, and then from there coming up with a volume based on the time step of the flow value.

To model this weir in MIKE URBAN, King County practice is to use the orifice function and set the coefficient equal to 1.

5.2 Hydraulic Verification

There were many challenges reconciling the differences between the sub-basin hydrographs, the upstream flow meter at the 11th Ave NW weir, and the reported overflow values at the 11th Ave NW weir. It was necessary to modify the model far from real-life conditions to try to match hydrographs while still coming reasonably close to these reported overflows. When this was occurring more data was requested downstream of the weir on the in-line side, as well as on the overflow side to explore what may be happening at the weir with more confidence.

The meter just downstream of the weir on route to the Ballard Regulator was installed at LU17-02 (SPU ID 012-169), approximately 300' downstream of the overflow weir, in 10/2016. This flow meter corroborated the flows recorded by the 11th Ave portable flow meter, just upstream of the weir. The subtraction of the data from these two meters indicated that the reported overflow values may be underestimating actual overflow volumes.

Following this, a meter in the overflow pipe was installed in 12/2016, approximately 30 downstream of the weir, on route to the ship canal. Not only did this meter show there were significantly higher overflows than reported, the overflow data matched the subtraction of the meters upstream and downstream of the weir.

The model was then modified to fit the new flow meter data. For the 8 overflow events where full flow meter data was obtained, the hydrology of upstream basins and hydraulics at the overflow structure were modified to match the hydrographs more effectively.

Because of these modifications, the hydrologic calibrations of the upstream basins were modified and now mostly carried bias to higher volumes. This was intentionally performed to best match the flows at the 11th Ave NW flow meter. With these much higher overflow volumes represented by all 3 flow meters at or near the 11th Ave NW weir, the overflow control volumes were found to be higher than was originally anticipated by the reported volumes of the past several years.

Table 5-A shows the final hydraulic orifice parameters for MIKE URBAN:

Table 5-A: MIKE URBAN Orifice parameters for 11th Ave NW Weir

MIKE URBAN Orifice Properties		
Weir Type:	Rectangular Orifice	
Oper. Mode:	No Control	
Weir Crest Elevation:	118.16	ft
Flap:	FALSE	
Discharge coefficient:	1.00	
Height from weir crest to vault ceiling:	2.74	ft
Width of weir:	29.43	ft

6 Results

6.1 Hydrologic Calibration

The calibrated parameters for each sub-basin are given in Table 6-A. For each calibrated basin, the evaluation statistics are presented in the table below. Graphs showing the calibrated model output, observed flow, and precipitation for each sub-basin are included in Appendix A. Graphs for the hydraulic verification are provided in Appendix B.

Table 6-A Calibrated Parameters and Goodness-of-Fit Metrics for each Sub-Basin

Parameters	BALL_4	BALL_5	BALL_6_7	BALL_08_14	BALL_9	BALL_13	BALL_15
RDII_AREA	25.0%	29.7%	8.10%	100%	18.2%	30.0%	6.54%
B_A_IFlat	34.13 %	32.10 %	10.0 %	100 %	11.0 %	27.0 %	9.50 %
B_M_IFlat	0.7350	0.1685	0.0319	0.4815	0.01	0.4815	0.3109
UMAX	3.25 in	1.99 in	0.003 in	0.84 in	0.001 in	0.04 in	2.62 in
LMAX	3.67 in	6.404 in	55.78 in	0.15 in	2.05 in	0.60 in	3.83 in
CQOF	0.502	0.110	0.010	0.700	0.010	0.132	0.782
CK	4.59 hr	5.26 hr	9.14 hr	3.80 hr	7.86 hr	3.46 hr	10.58 hr
CKIF	953.5 hr	392.76 hr	538.47 hr	26.1 hr	999.7 hr	63.1 hr	382.2 hr
CKBF	2097.3 hr	1456.9 hr	2500.0 hr	817.6 hr	1884.0 hr	67.7 hr	1000.0 hr

BALL_004

Event	Bias	RMSE	Nash	Q_{pk}	V_{tot}
01	17.3%	1.14	0.82	-9.3%	8.9%
02	12.5%	1.22	0.84	7.5%	9.4%
03	17.2%	0.77	0.73	10.0%	43.2%
04	17.9%	1.06	0.73	-29.9%	25.4%
05	3.2%	0.06	0.72	-10.2%	-1.5%
06	2.9%	0.04	0.86	-9.0%	-1.8%
07	40.2%	1.01	0.75	-0.3%	36.0%
08	39.2%	0.78	0.64	4.1%	49.4%
09	16.3%	0.94	0.85	21.5%	25.7%
10	11.4%	0.81	0.72	-20.8%	10.1%

BALL_005

Event	Bias	RMSE	Nash	Q_{pk}	V_{tot}
01	6.2%	0.54	0.96	3.0%	1.7%
02	-1.0%	0.37	0.96	5.5%	8.2%
03	1.5%	0.05	0.90	-9.8%	0.9%
04	19.5%	0.43	0.73	23.3%	27.4%
05	9.7%	0.24	0.90	-3.7%	7.3%
06	7.0%	0.49	0.95	-6.4%	2.1%
07	1.5%	0.55	0.92	-17.8%	-1.8%
08	6.0%	0.49	0.90	-15.5%	5.5%
09	-4.0%	0.42	0.97	-17.4%	-0.4%
10	-3.8%	0.93	0.86	-11.0%	-6.4%

BALL_006_007

Event	Bias	RMSE	Nash	Q_{pk}	V_{tot}
01	-9.0%	1.11	0.38	-30.7%	-5.2%
02	-5.3%	5.19	0.42	-27.7%	-40.3%
03	5.5%	1.14	0.78	17.5%	13.8%
04	5.5%	1.59	0.77	-6.8%	7.2%
05	-2.0%	1.20	0.89	7.4%	7.6%
06	-5.9%	1.14	0.58	-32.1%	0.9%
07	-4.7%	1.28	0.84	8.8%	-1.9%
08	-12.8%	1.26	0.59	-17.2%	8.0%
09	-16.7%	1.18	0.09	-0.4%	-8.7%
10	-16.9%	1.02	-0.74	-13.3%	-13.6%

BALL_009

Event	Bias	RMSE	Nash	Q_{pk}	V_{tot}
01	5.1%	0.08	0.86	-6.0%	76.6%
02	25.1%	1.07	0.27	21.5%	42.9%
03	16.6%	1.46	0.67	2.4%	22.8%
04	16.7%	0.81	0.81	-5.0%	15.0%
05	38.4%	1.81	0.45	1.4%	63.0%
06	1.6%	1.01	0.89	17.2%	4.8%
07	36.4%	0.90	0.36	-23.6%	40.0%
08	82.0%	2.24	-0.75	-9.9%	97.0%
09	38.6%	1.02	0.46	-34.2%	48.2%
10	17.4%	0.29	-1.21	64.1%	20.2%

BALL_015

Event	Bias	RMSE	Nash	Q_{pk}	V_{tot}
01	15.2%	0.06	0.79	8.1%	17.7%
02	7.7%	0.07	0.36	-53.3%	7.4%
03	23.7%	0.07	0.27	35.4%	39.0%
04	36.7%	0.04	0.57	-8.3%	31.2%
05	35.6%	0.04	0.60	-5.1%	44.8%
06	17.6%	0.06	0.89	4.0%	16.3%
07	11.8%	0.05	0.90	-21.9%	12.1%
08	8.6%	0.04	0.86	-12.4%	13.1%
09	11.7%	0.06	0.84	4.6%	22.2%
10	3.0%	0.09	0.70	-20.2%	8.4%

6.1.1 Calibration Graphs

Appendix A contains calibration graphs that show the simulated flow, flow meter, and rain gauge time series used to calibrate each sub-basin, as well as the calibrated hydrologic parameters and various goodness-of-fit metrics.

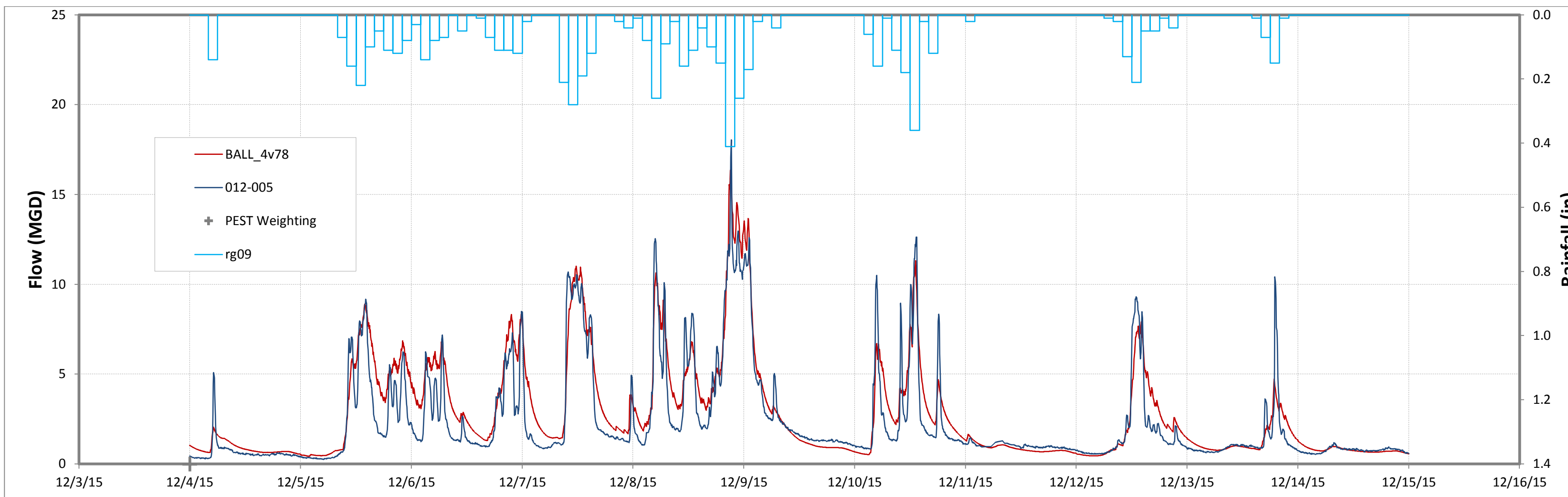
6.2 Hydraulic Verification

6.2.1 Verification Graphs

Appendix B contains graphs that show MIKE Urban models, both pre-flow meter installations at 11th Ave Weir and post-installation, as well as the measured flow values for the following conditions: Upstream of the 11th Ave NW Weir structure (LU16-01A), downstream of the overflow structure on the underflow side (LU17-02), and on the overflow side. The Overflow graph shows both the subtraction of flow meters upstream and downstream of the weir as well as the measured values from the meter installed on the overflow side of the weir structure.

Appendix A: Hydrologic Calibrations

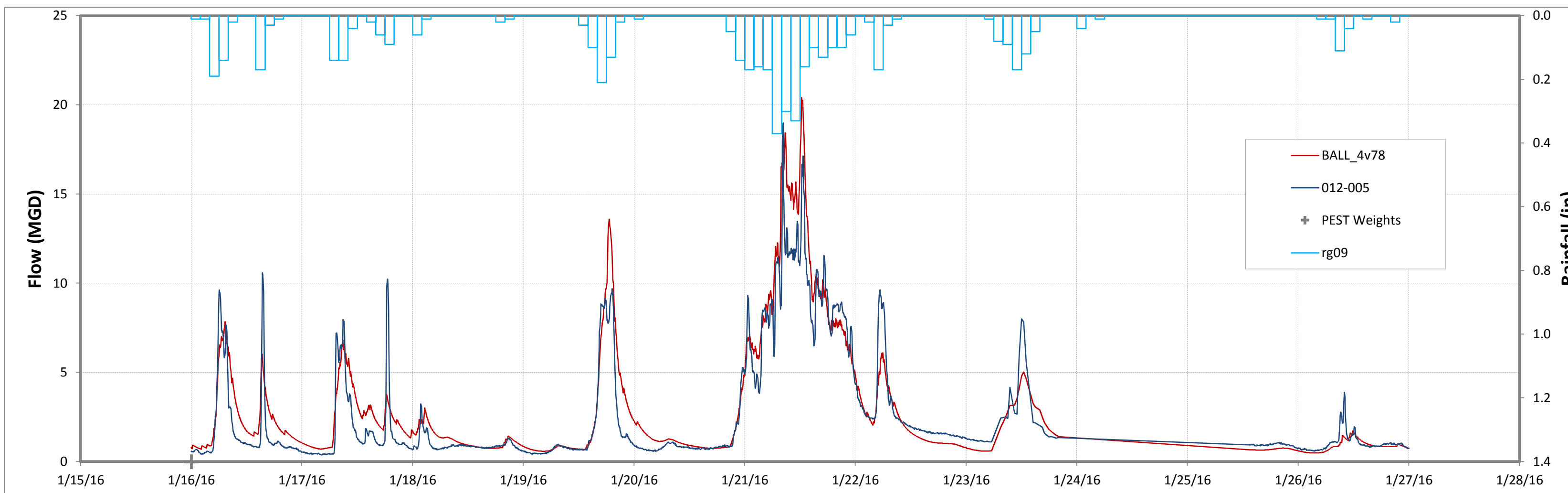
11th Ave NW CSO Model Development and Calibration Report



Event 01
12/04/15 to 12/15/15

PEST Optimization	
RDII_AREA	25.0 %
B_A_IFLAT	34.1 %
B_M_IFLAT	1.36
UMAX	82.62 mm
LMAX	93.09 mm
CQOF	0.50
CK	4.6 hr
CKIF	953.5 hr
CKBF	2097.3 hr

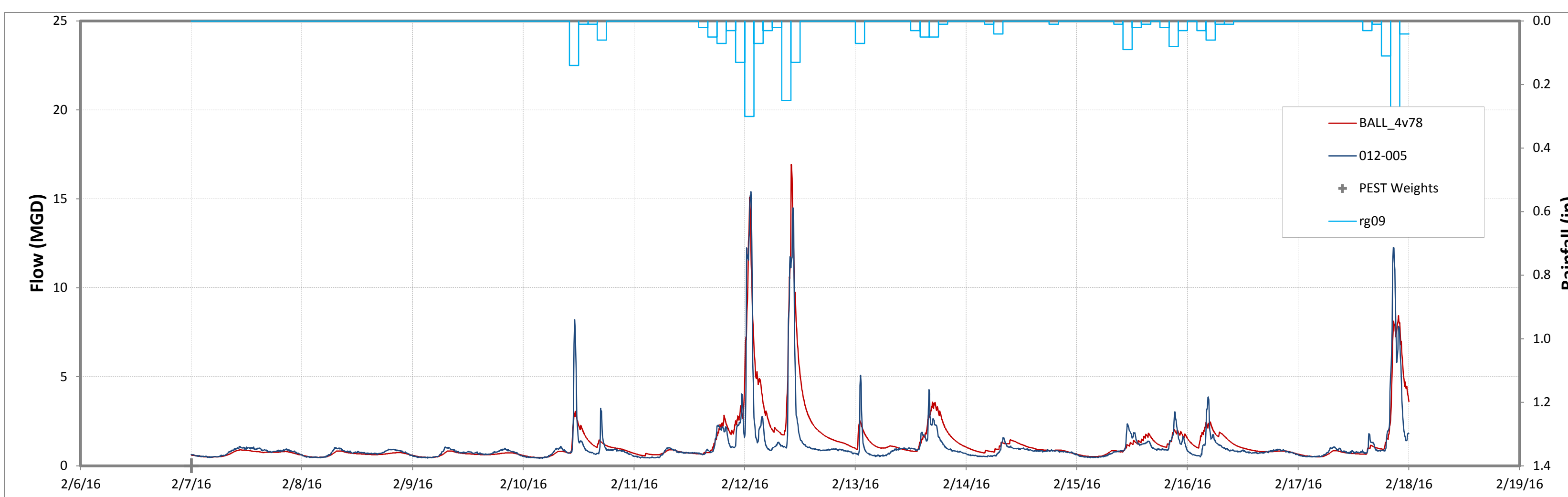
Goodness-of-Fit	
Threshold	0.00
Bias	17.25%
RMSE	1.14
Nash	0.82
Q_{pk} Error	-9.27%
V_{tot} Error	8.86%



Event 02
01/16/16 to 01/27/16

PEST Optimization	
RDII_AREA	25.0 %
B_A_IFLAT	34.1 %
B_M_IFLAT	1.36
UMAX	82.62 mm
LMAX	93.09 mm
CQOF	0.50
CK	4.6 hr
CKIF	953.5 hr
CKBF	2097.3 hr

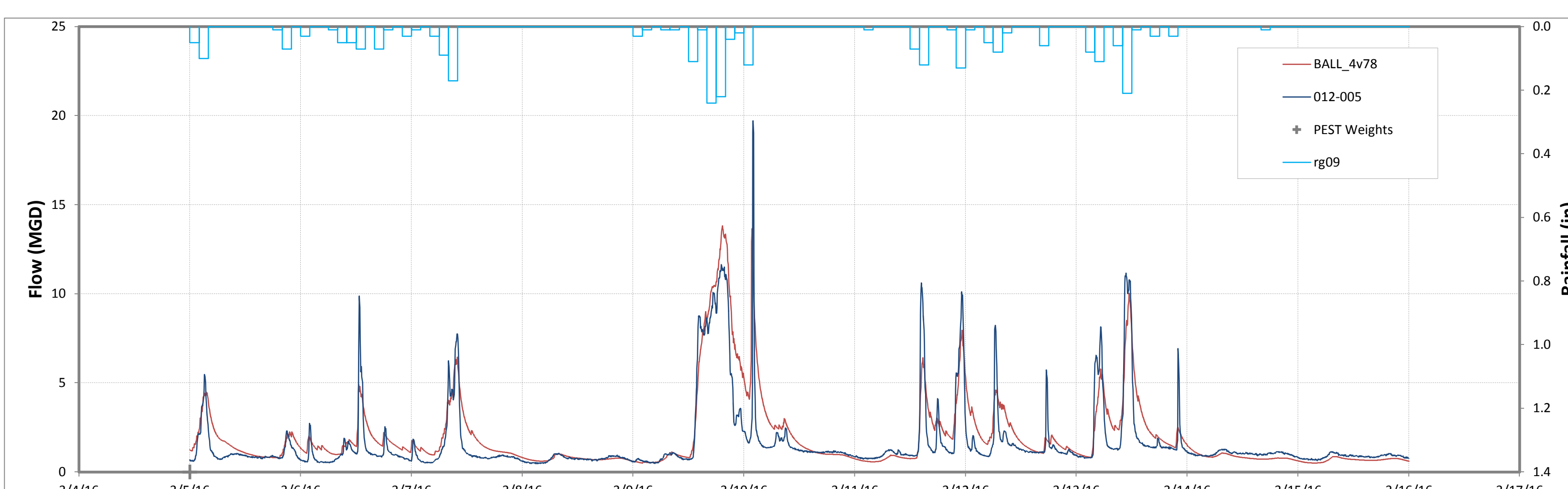
Goodness-of-Fit	
Threshold	0.00
Bias	12.51%
RMSE	1.22
Nash	0.84
Q_{pk} Error	7.48%
V_{tot} Error	9.41%



Event 03
02/07/16 to 02/18/16

PEST Optimization	
RDII_AREA	25.0 %
B_A_IFLAT	34.1 %
B_M_IFLAT	1.36
UMAX	82.62 mm
LMAX	93.09 mm
CQOF	0.50
CK	4.6 hr
CKIF	953.5 hr
CKBF	2097.3 hr

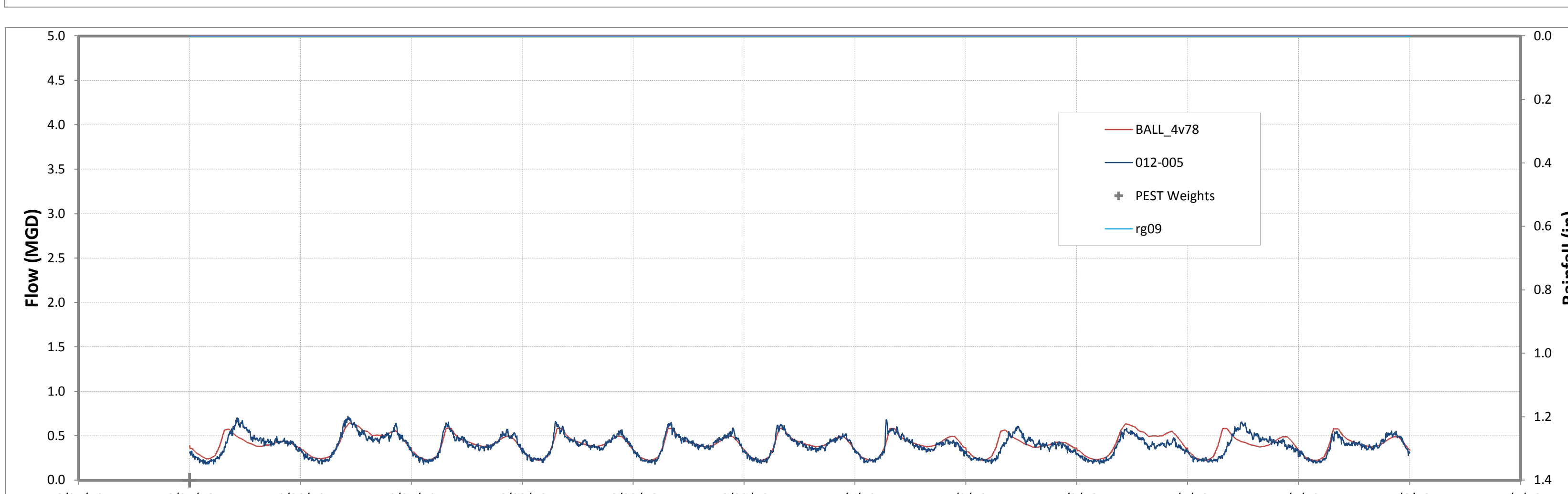
Goodness-of-Fit	
Threshold	0.00
Bias	17.22%
RMSE	0.77
Nash	0.73
Q_{pk} Error	9.97%
V_{tot} Error	43.25%



Event 04
03/05/16 to 03/16/16

PEST Optimization	
RDII_AREA	25.0 %
B_A_IFLAT	34.1 %
B_M_IFLAT	1.36
UMAX	82.62 mm
LMAX	93.09 mm
CQOF	0.50
CK	4.6 hr
CKIF	953.5 hr
CKBF	2097.3 hr

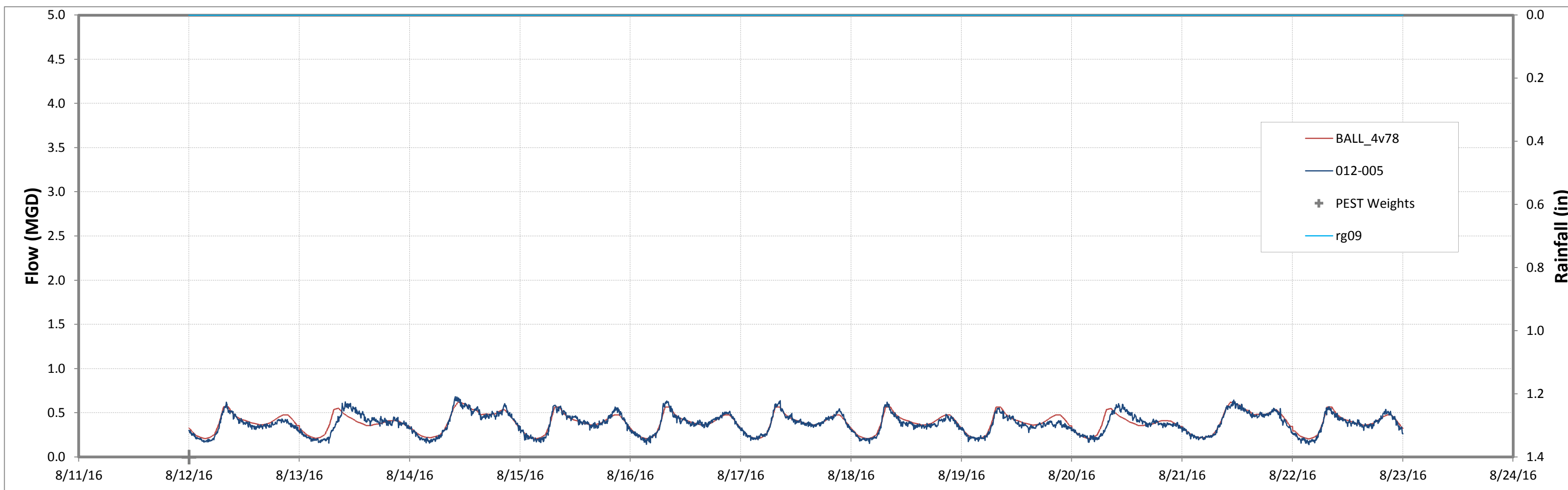
Goodness-of-Fit	
Threshold	0.00
Bias	17.85%
RMSE	1.06
Nash	0.73
Q_{pk} Error	-29.90%
V_{tot} Error	25.36%



Event 05
06/25/16 to 07/06/16

PEST Optimization	
RDII_AREA	25.0 %
B_A_IFLAT	34.1 %
B_M_IFLAT	1.36
UMAX	82.62 mm
LMAX	93.09 mm
CQOF	0.50
CK	4.6 hr
CKIF	953.5 hr
CKBF	2097.3 hr

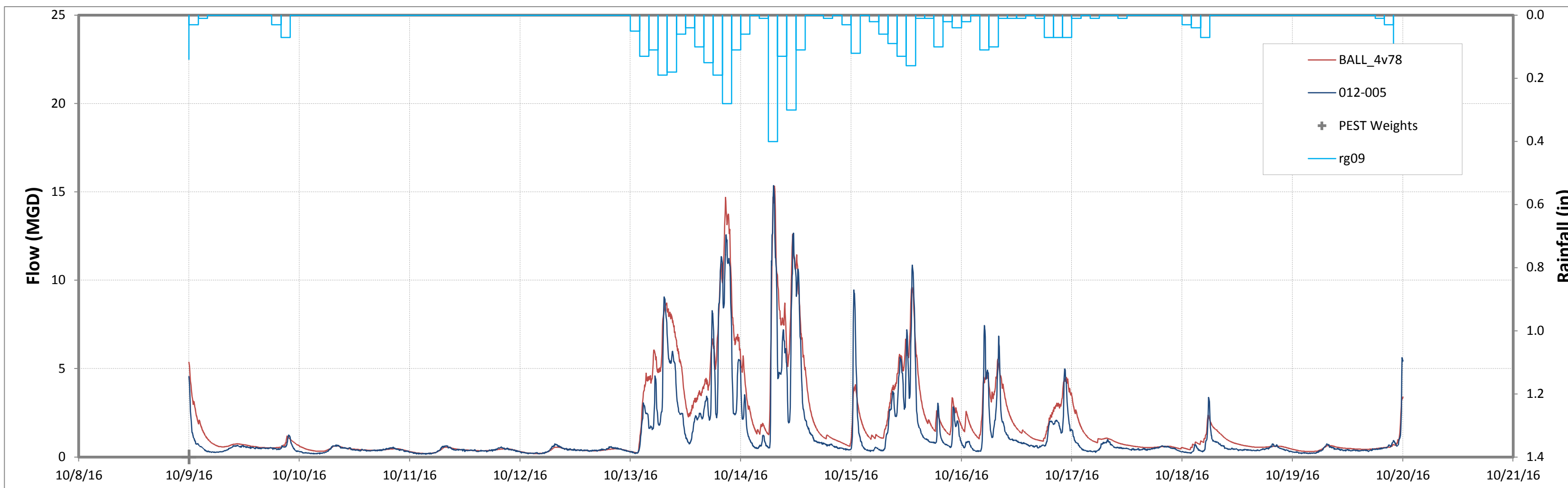
Goodness-of-Fit	
Threshold	0.00
Bias	3.20%
RMSE	0.06
Nash	0.72
Q_{pk} Error	-10.24%
V_{tot} Error	-1.55%



Event 06
08/12/16 to 08/23/16

PEST Optimization	
RDII_AREA	25.0 %
B_A_IFLAT	34.1 %
B_M_IFLAT	1.36
UMAX	82.62 mm
LMAX	93.09 mm
CQOF	0.50
CK	4.6 hr
CKIF	953.5 hr
CKBF	2097.3 hr

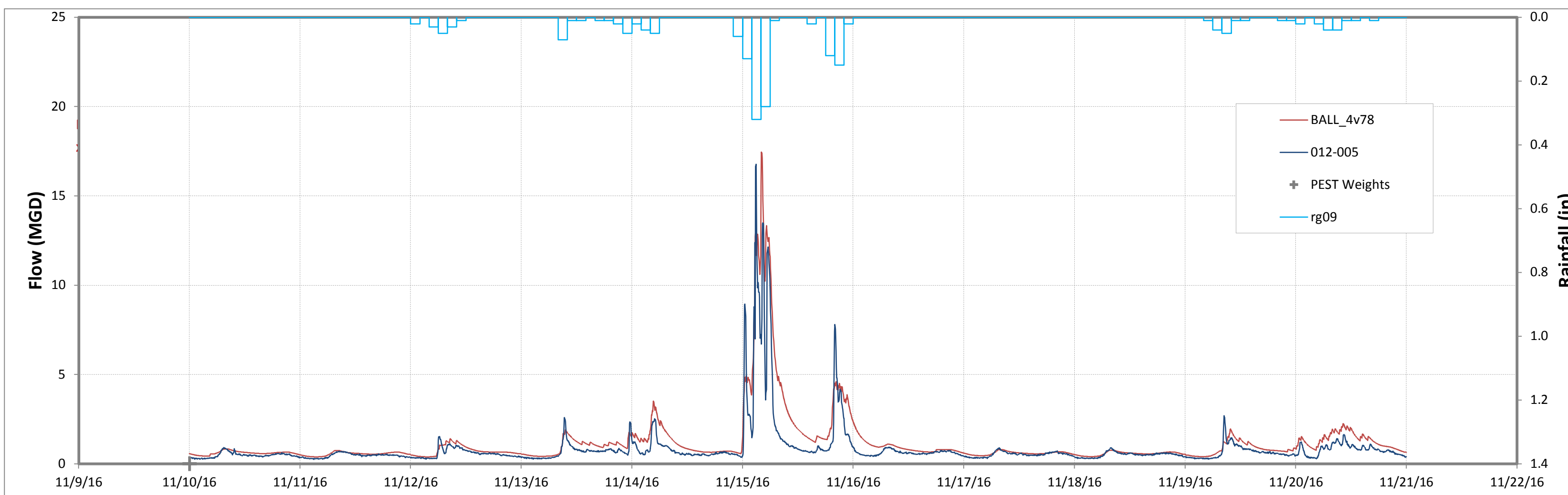
Goodness-of-Fit	
Threshold	0.00
Bias	2.91%
RMSE	0.04
Nash	0.86
Q_{pk} Error	-9.02%
V_{tot} Error	-1.85%



Event 07
10/09/16 to 10/20/16

PEST Optimization	
RDII_AREA	25.0 %
B_A_IFLAT	34.1 %
B_M_IFLAT	1.36
UMAX	82.62 mm
LMAX	93.09 mm
CQOF	0.50
CK	4.6 hr
CKIF	953.5 hr
CKBF	2097.3 hr

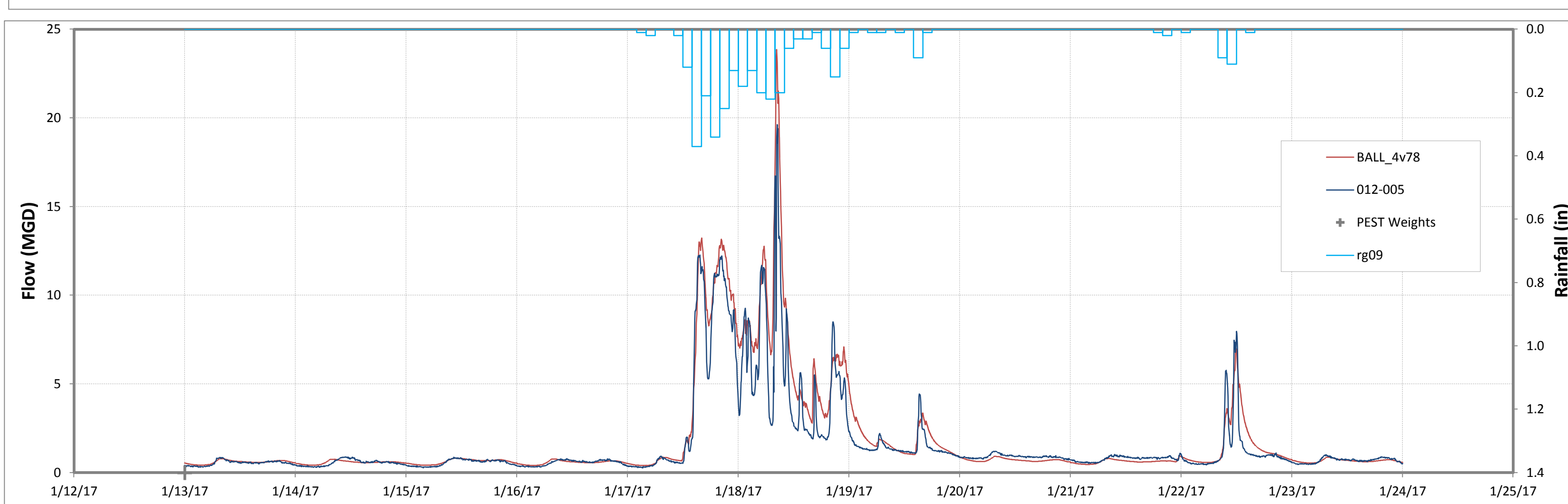
Goodness-of-Fit	
Threshold	0.00
Bias	40.16%
RMSE	1.01
Nash	0.75
Q_{pk} Error	-0.31%
V_{tot} Error	36.00%



Event 08
11/10/16 to 11/21/16

PEST Optimization	
RDII_AREA	25.0 %
B_A_IFLAT	34.1 %
B_M_IFLAT	1.36
UMAX	82.62 mm
LMAX	93.09 mm
CQOF	0.50
CK	4.6 hr
CKIF	953.5 hr
CKBF	2097.3 hr

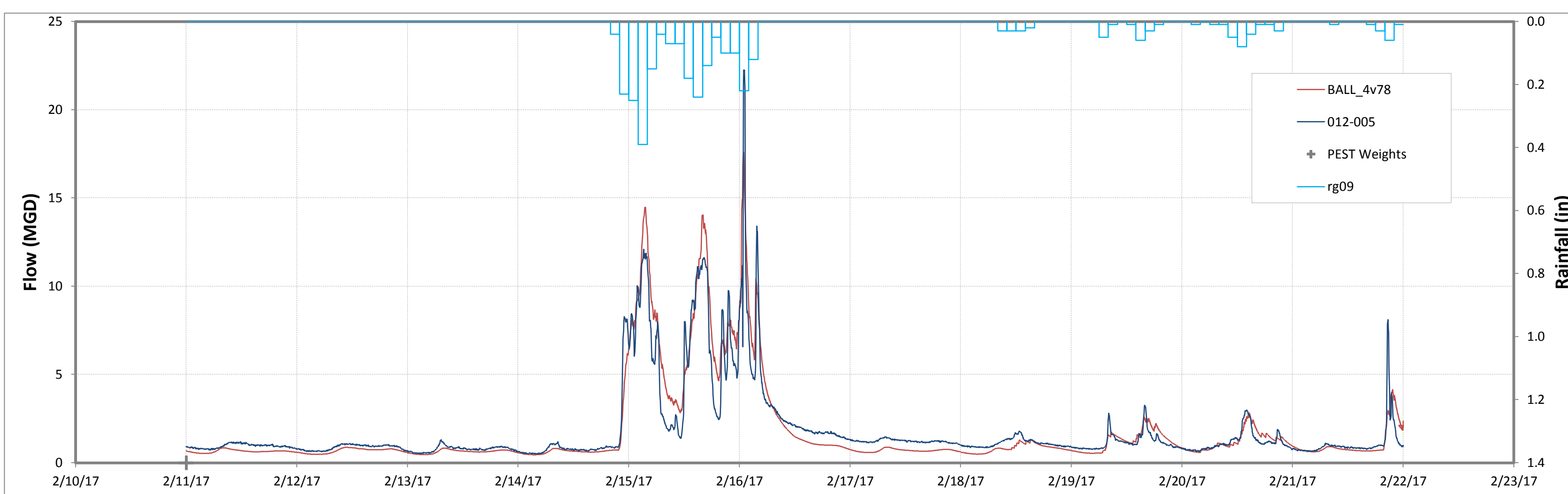
Goodness-of-Fit	
Threshold	0.00
Bias	39.24%
RMSE	0.78
Nash	0.643
Q_{pk} Error	4.08%
V_{tot} Error	49.38%



Event 09
01/13/17 to 01/24/17

PEST Optimization	
RDII_AREA	25.0 %
B_A_IFLAT	34.1 %
B_M_IFLAT	1.36
UMAX	82.62 mm
LMAX	93.09 mm
CQOF	0.50
CK	4.6 hr
CKIF	953.5 hr
CKBF	2097.3 hr

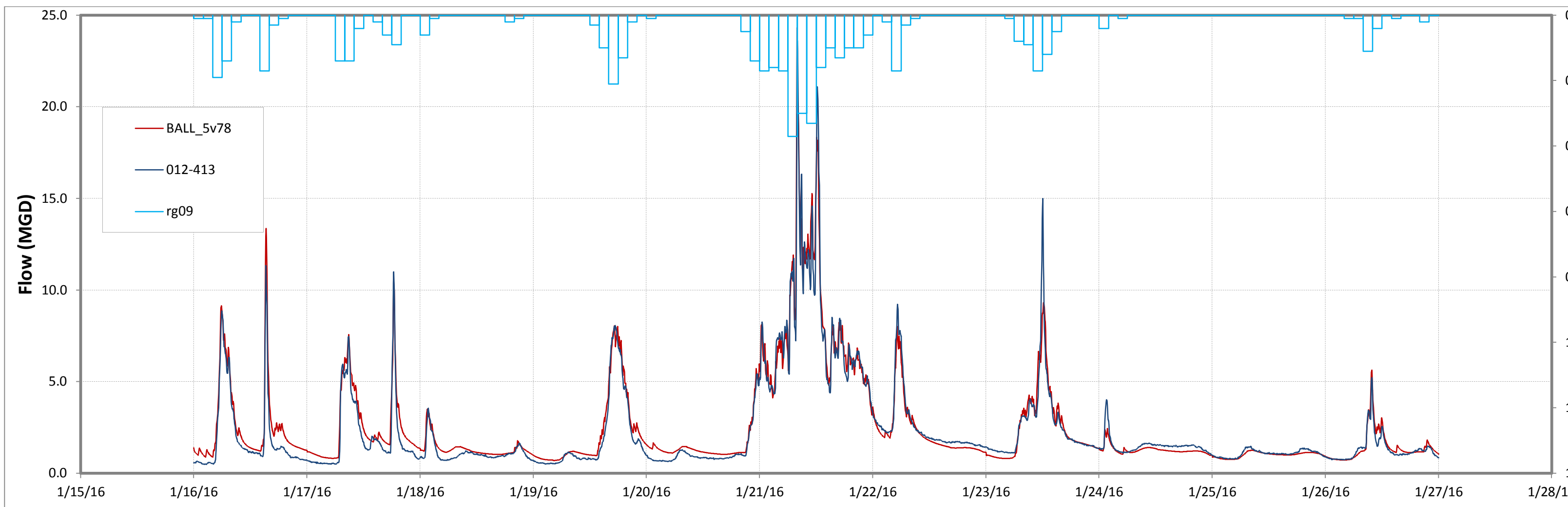
Goodness-of-Fit	
Threshold	0.00
Bias	16.29%
RMSE	0.94
Nash	0.85
Q_{pk} Error	21.55%
V_{tot} Error	25.71%



Event 10
02/11/17 to 02/22/17

PEST Optimization	
RDII_AREA	25.0 %
B_A_IFLAT	34.1 %
B_M_IFLAT	1.36
UMAX	82.62 mm
LMAX	93.09 mm
CQOF	0.50
CK	4.6 hr
CKIF	953.5 hr
CKBF	2097.3 hr

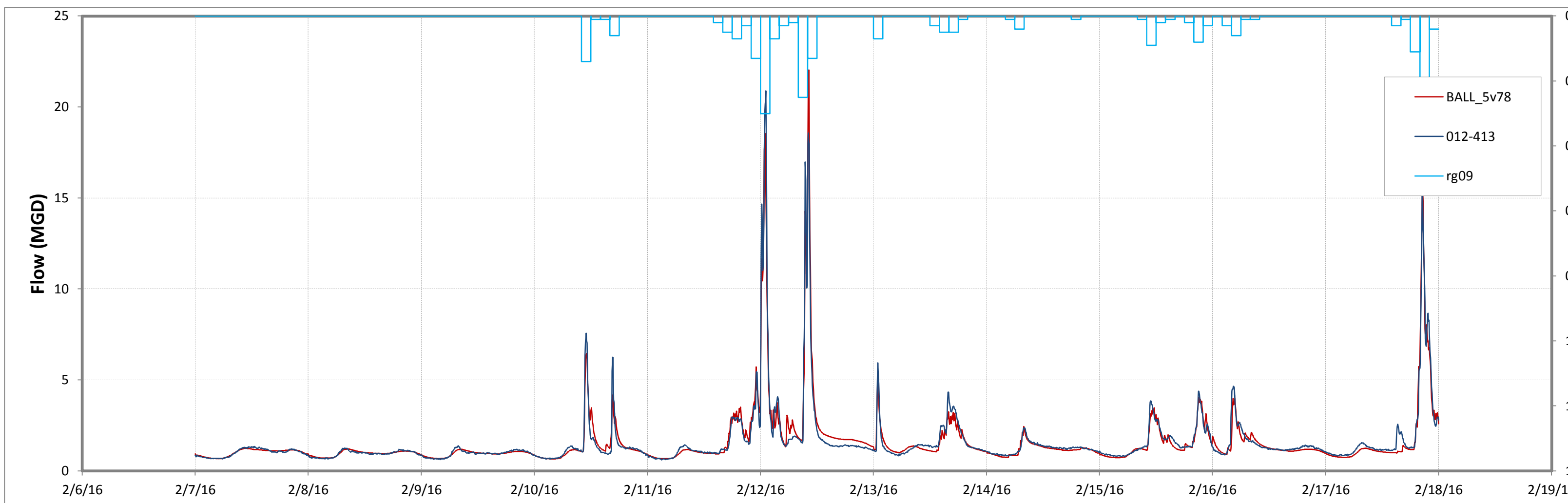
Goodness-of-Fit	
Threshold	0.00
Bias	#N/A
RMSE	#N/A
Nash	#N/A
Q_{pk} Error	-20.83%
V_{tot} Error	10.15%



Event 01
01/16/16 to 01/27/16

PEST Optimization	
RDII_AREA	29.7 %
B_A_IFLAT	32.1 %
B_M_IFLAT	5.93
UMAX	50.68 mm
LMAX	162.67 mm
CQOF	0.41
CK	5.3 hr
CKIF	392.8 hr
CKBF	1456.9 hr

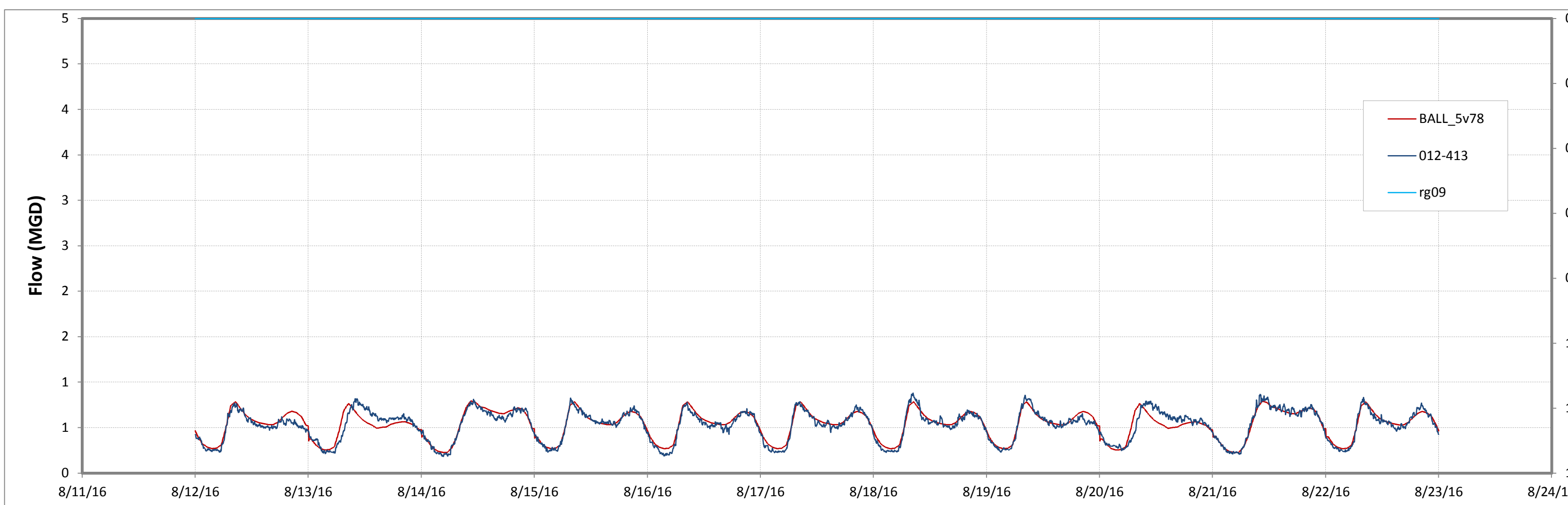
Goodness-of-Fit	
Threshold	0.00
Bias	6.24%
RMSE	0.54
Nash	0.96
Q_{pk} Error	2.97%
V_{tot} Error	1.66%



Event 02
02/07/16 to 02/18/16

PEST Optimization	
RDII_AREA	29.7 %
B_A_IFLAT	32.1 %
B_M_IFLAT	5.93
UMAX	50.68 mm
LMAX	162.67 mm
CQOF	0.41
CK	5.3 hr
CKIF	392.8 hr
CKBF	1456.9 hr

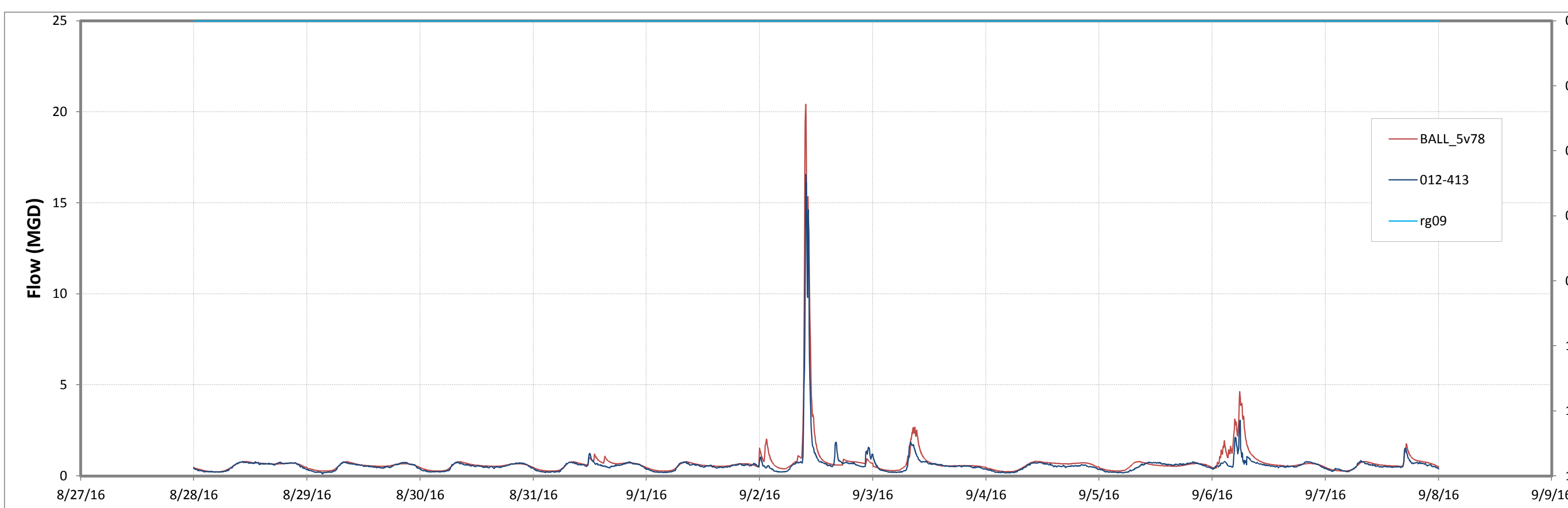
Goodness-of-Fit	
Threshold	0.00
Bias	-1.03%
RMSE	0.37
Nash	0.96
Q_{pk} Error	5.52%
V_{tot} Error	8.17%



Event 03
08/12/16 to 08/23/16

PEST Optimization	
RDII_AREA	29.7 %
B_A_IFLAT	32.1 %
B_M_IFLAT	5.93
UMAX	50.68 mm
LMAX	162.67 mm
CQOF	0.41
CK	5.3 hr
CKIF	392.8 hr
CKBF	1456.9 hr

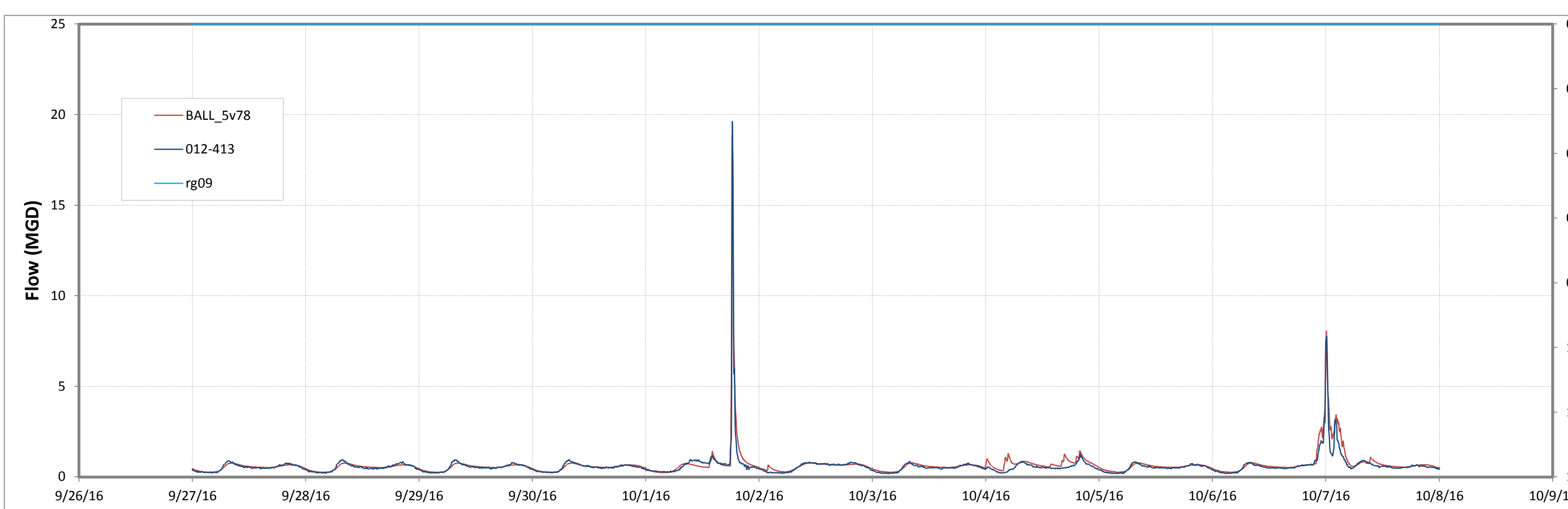
Goodness-of-Fit	
Threshold	0.00
Bias	1.47%
RMSE	0.05
Nash	0.90
Q_{pk} Error	-9.84%
V_{tot} Error	0.93%



Event 04
08/28/16 to 09/08/16

PEST Optimization	
RDII_AREA	29.7 %
B_A_IFLAT	32.1 %
B_M_IFLAT	5.93
UMAX	50.68 mm
LMAX	162.67 mm
CQOF	0.41
CK	5.3 hr
CKIF	392.8 hr
CKBF	1456.9 hr

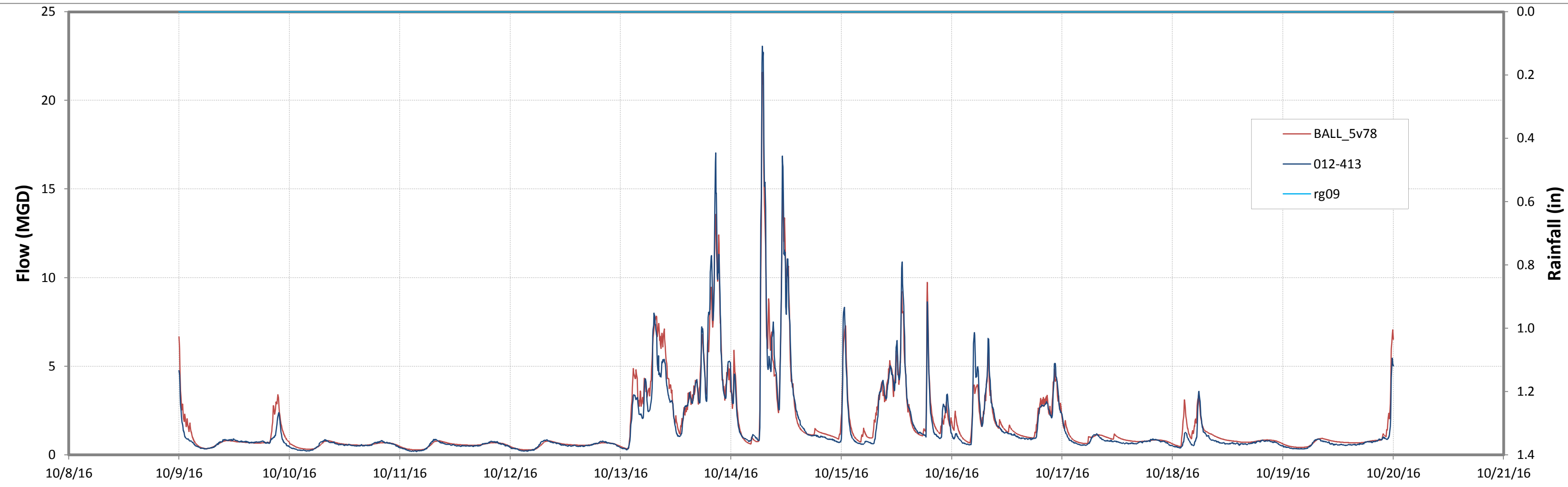
Goodness-of-Fit	
Threshold	0.00
Bias	19.49%
RMSE	0.43
Nash	0.73
Q_{pk} Error	23.29%
V_{tot} Error	27.42%



Event 05
09/27/16 to 10/08/16

PEST Optimization	
RDII_AREA	29.7 %
B_A_IFLAT	32.1 %
B_M_IFLAT	5.93
UMAX	50.68 mm
LMAX	162.67 mm
CQOF	0.41
CK	5.3 hr
CKIF	392.8 hr
CKBF	1456.9 hr

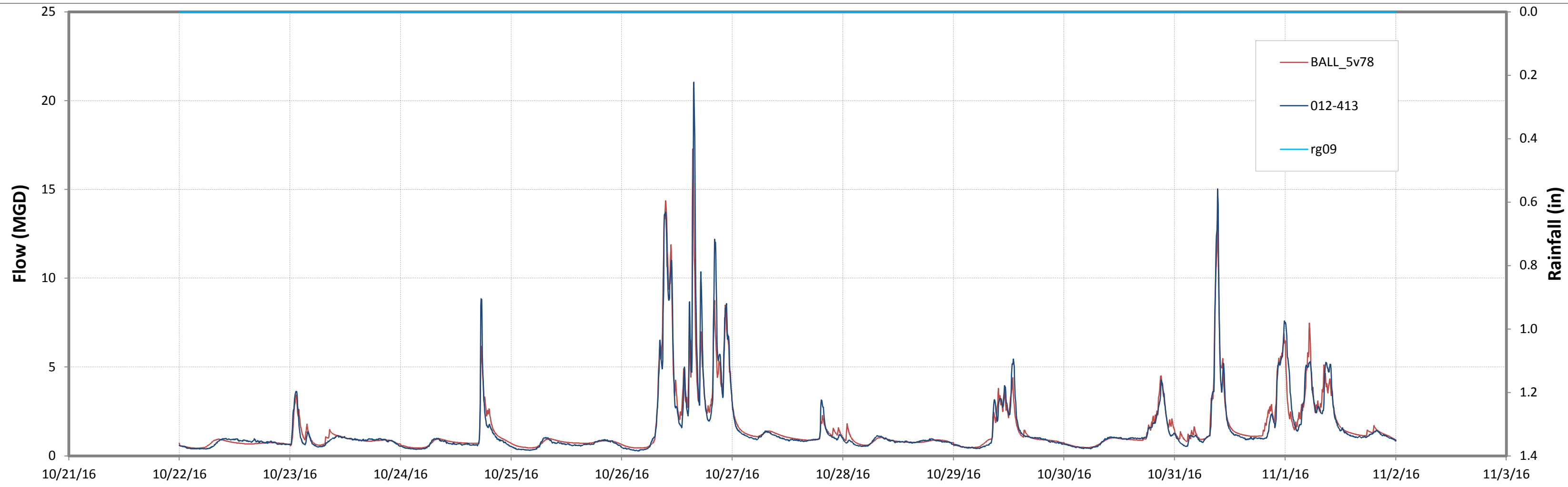
Goodness-of-Fit	
Threshold	0.00
Bias	9.74%
RMSE	0.24
Nash	0.90
Q_{pk} Error	-3.74%
V_{tot} Error	7.26%



Event 06
10/09/16 to 10/20/16

PEST Optimization	
RDII_AREA	29.7 %
B_A_IFLAT	32.1 %
B_M_IFLAT	5.93
UMAX	50.68 mm
LMAX	162.67 mm
CQOF	0.41
CK	5.3 hr
CKIF	392.8 hr
CKBF	1456.9 hr

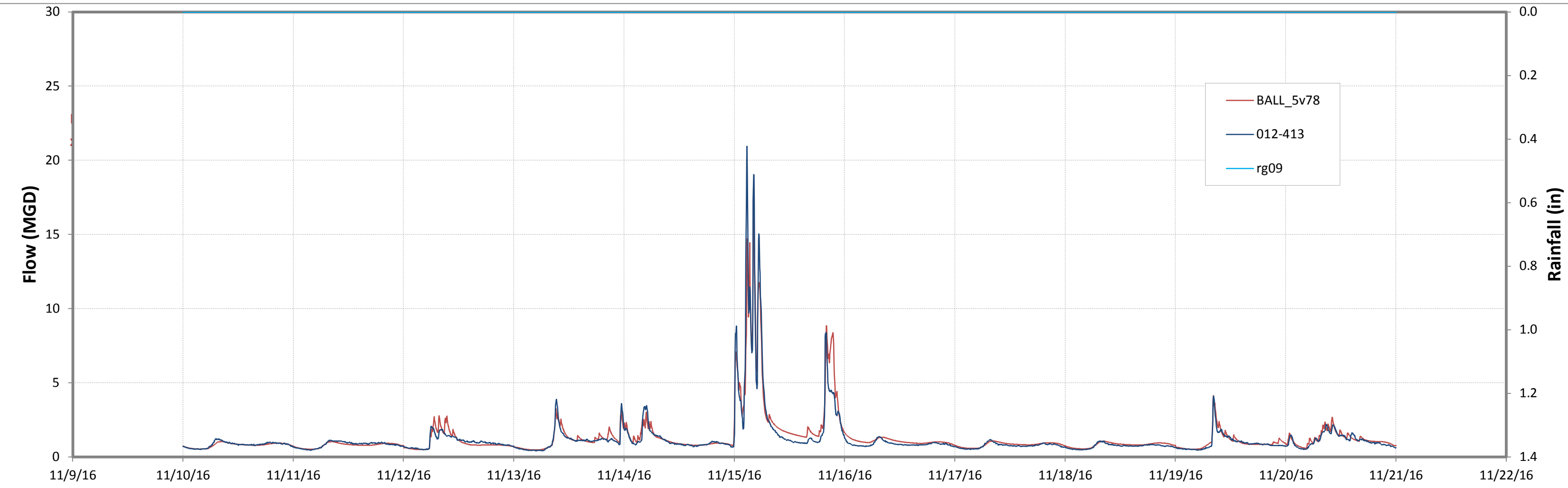
Goodness-of-Fit	
Threshold	0.00
Bias	7.03%
RMSE	0.49
Nash	0.95
Q_{pk} Error	-6.37%
V_{tot} Error	2.13%



Event 07
10/22/16 to 11/02/16

PEST Optimization	
RDII_AREA	29.7 %
B_A_IFLAT	32.1 %
B_M_IFLAT	5.93
UMAX	50.68 mm
LMAX	162.67 mm
CQOF	0.41
CK	5.3 hr
CKIF	392.8 hr
CKBF	1456.9 hr

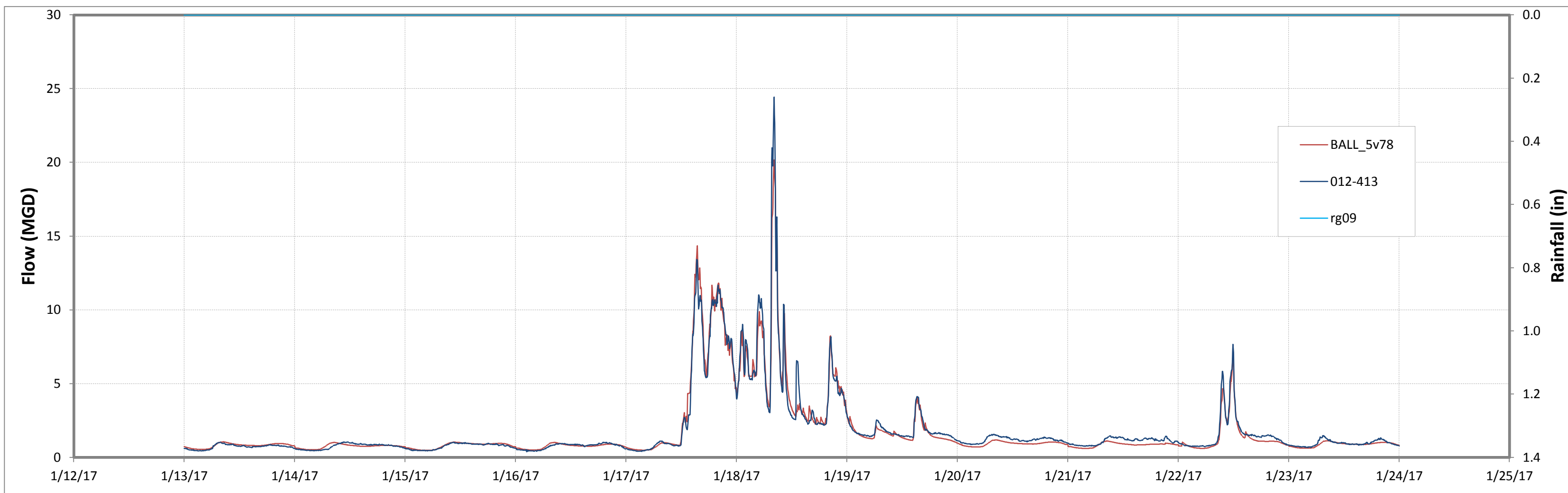
Goodness-of-Fit	
Threshold	0.00
Bias	1.55%
RMSE	0.55
Nash	0.92
Q_{pk} Error	-17.82%
V_{tot} Error	-1.84%



Event 08
11/10/16 to 11/21/16

PEST Optimization	
RDII_AREA	29.7 %
B_A_IFLAT	32.1 %
B_M_IFLAT	5.93
UMAX	50.68 mm
LMAX	162.67 mm
CQOF	0.41
CK	5.3 hr
CKIF	392.8 hr
CKBF	1456.9 hr

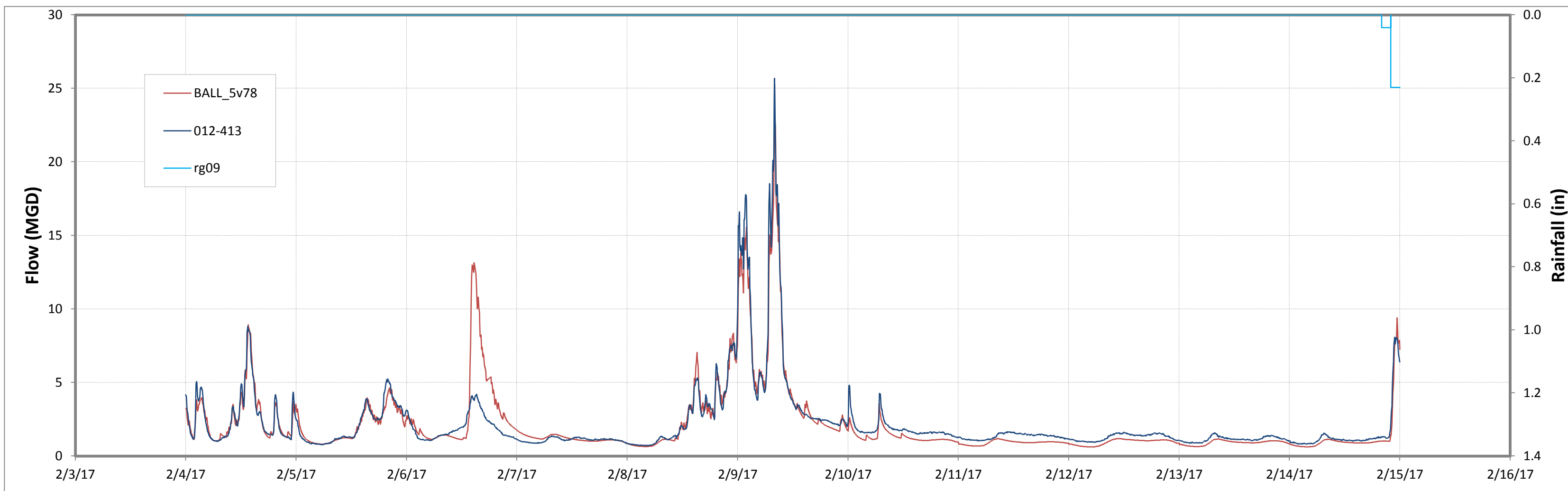
Goodness-of-Fit	
Threshold	0.00
Bias	6.04%
RMSE	0.49
Nash	0.897
Q_{pk} Error	-15.51%
V_{tot} Error	5.47%



Event 09
01/13/17 to 01/24/17

PEST Optimization	
RDII_AREA	29.7 %
B_A_IFLAT	32.1 %
B_M_IFLAT	5.93
UMAX	50.68 mm
LMAX	162.67 mm
CQOF	0.41
CK	5.3 hr
CKIF	392.8 hr
CKBF	1456.9 hr

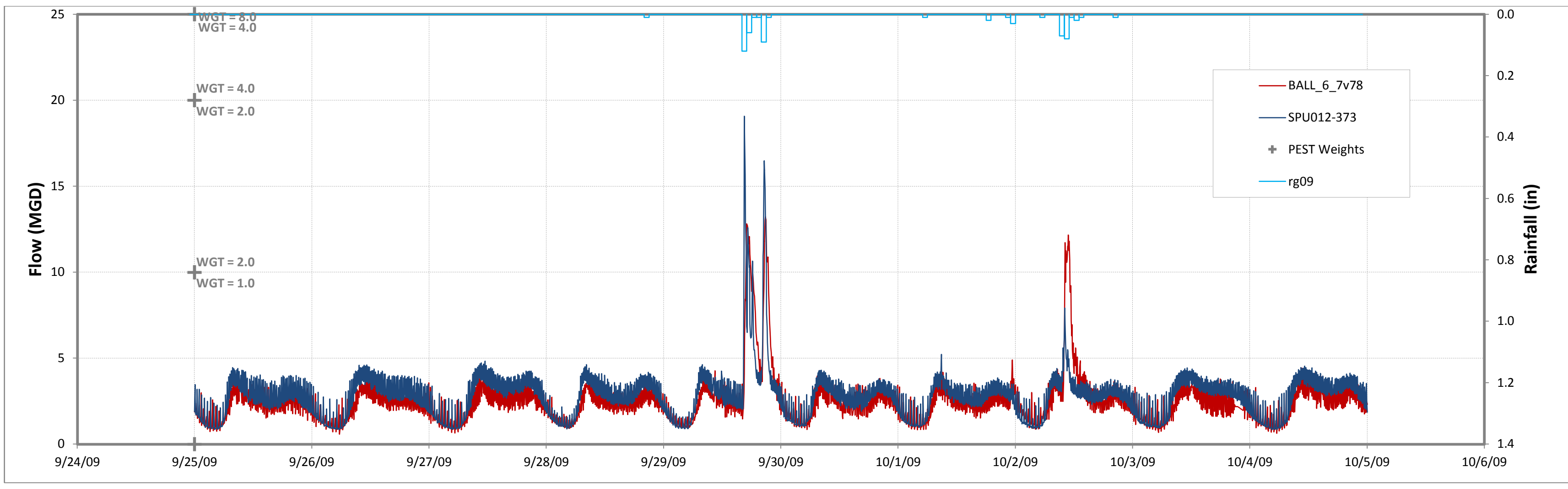
Goodness-of-Fit	
Threshold	0.00
Bias	-3.95%
RMSE	0.42
Nash	0.97
Q_{pk} Error	-17.44%
V_{tot} Error	-0.36%



Event 10
02/04/17 to 02/15/17

PEST Optimization	
RDII_AREA	29.7 %
B_A_IFLAT	32.1 %
B_M_IFLAT	5.93
UMAX	50.68 mm
LMAX	162.67 mm
CQOF	0.41
CK	5.3 hr
CKIF	392.8 hr
CKBF	1456.9 hr

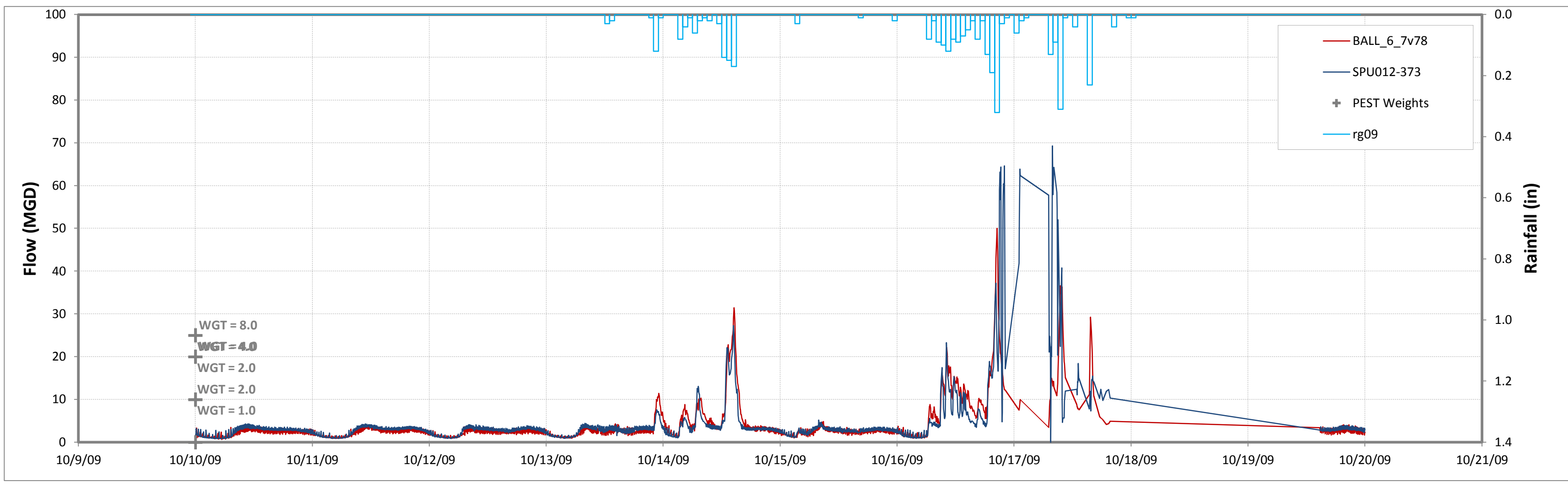
Goodness-of-Fit	
Threshold	0.00
Bias	-3.81%
RMSE	0.93
Nash	0.86
Q_{pk} Error	-11.01%
V_{tot} Error	-6.41%



Event 01
09/25/09 to 10/05/09

PEST Optimization	
RDII_AREA	8.1 %
B_A_IFLAT	10.0 %
B_M_IFLAT	0.03
UMAX	0.00 mm
LMAX	55.77 mm
CQOF	0.01
CK	9.1 hr
CKIF	538.5 hr
CKBF	2500.0 hr

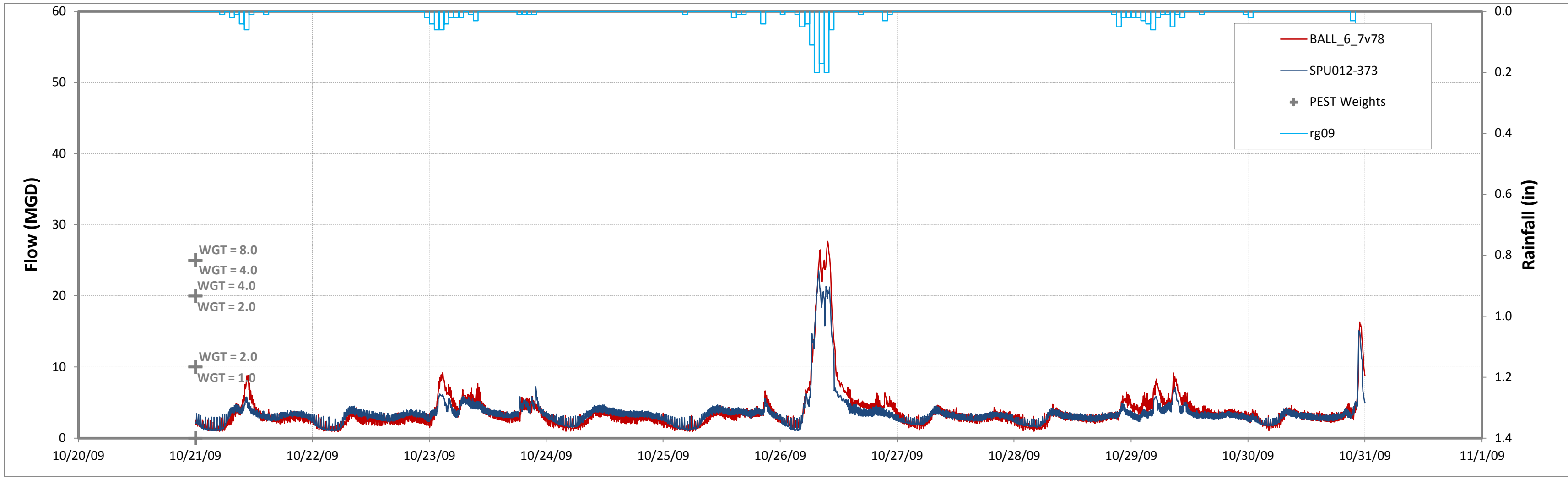
Goodness-of-Fit	
Threshold	0.00
Bias	-9.02%
RMSE	1.11
Nash	0.38
Q_{pk} Error	-30.69%
V_{tot} Error	-5.18%



Event 02
10/10/09 to 10/20/09

PEST Optimization	
RDII_AREA	8.1 %
B_A_IFLAT	10.0 %
B_M_IFLAT	0.03
UMAX	0.00 mm
LMAX	55.77 mm
CQOF	0.01
CK	9.1 hr
CKIF	538.5 hr
CKBF	2500.0 hr

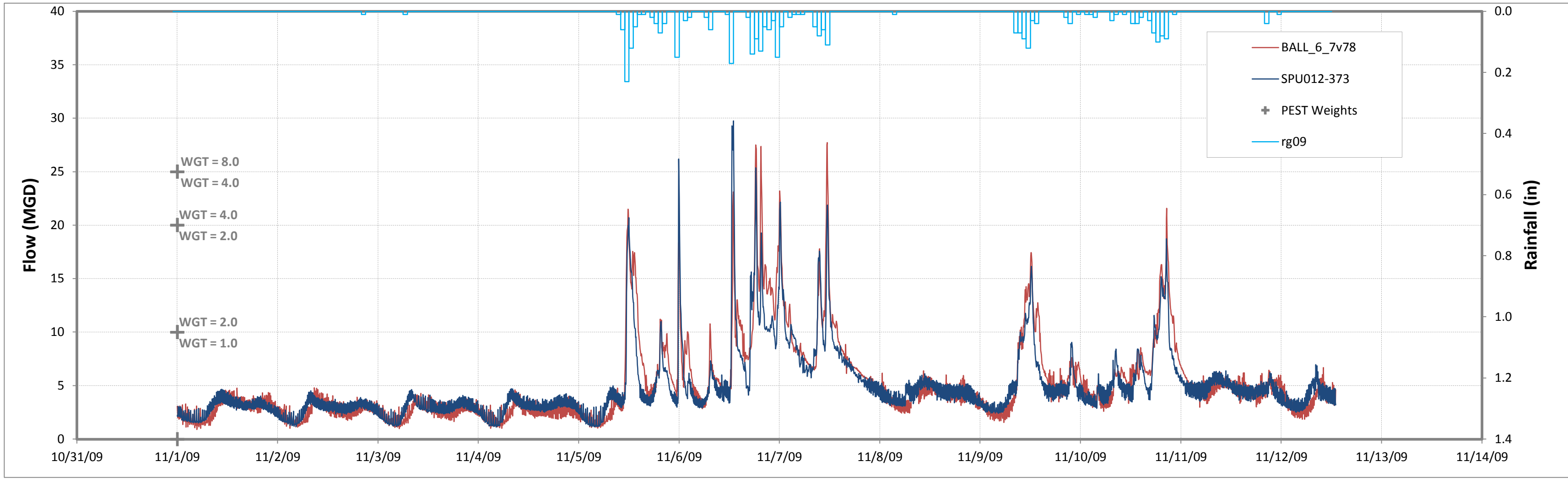
Goodness-of-Fit	
Threshold	0.00
Bias	-5.35%
RMSE	5.19
Nash	0.42
Q_{pk} Error	-27.74%
V_{tot} Error	-40.28%



Event 03
10/21/09 to 10/31/09

PEST Optimization	
RDII_AREA	8.1 %
B_A_IFLAT	10.0 %
B_M_IFLAT	0.03
UMAX	0.00 mm
LMAX	55.77 mm
CQOF	0.01
CK	9.1 hr
CKIF	538.5 hr
CKBF	2500.0 hr

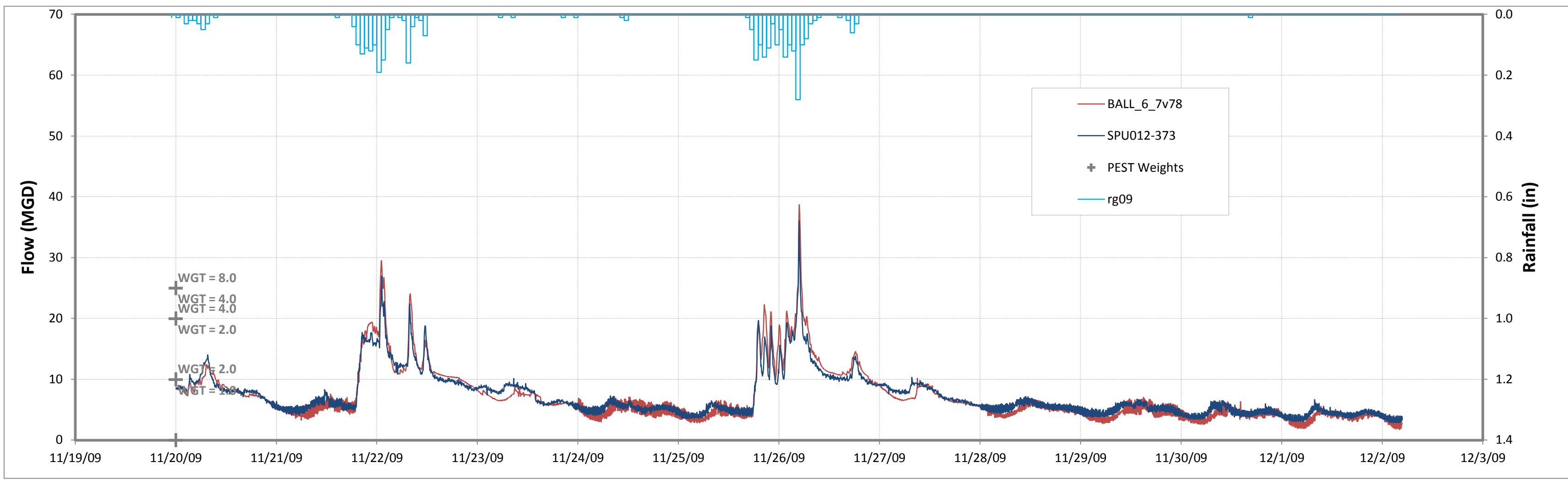
Goodness-of-Fit	
Threshold	0.00
Bias	5.46%
RMSE	1.14
Nash	0.78
Q_{pk} Error	17.45%
V_{tot} Error	13.77%



Event 04
11/01/09 to 11/12/09

PEST Optimization	
RDII_AREA	8.1 %
B_A_IFLAT	10.0 %
B_M_IFLAT	0.03
UMAX	0.00 mm
LMAX	55.77 mm
CQOF	0.01
CK	9.1 hr
CKIF	538.5 hr
CKBF	2500.0 hr

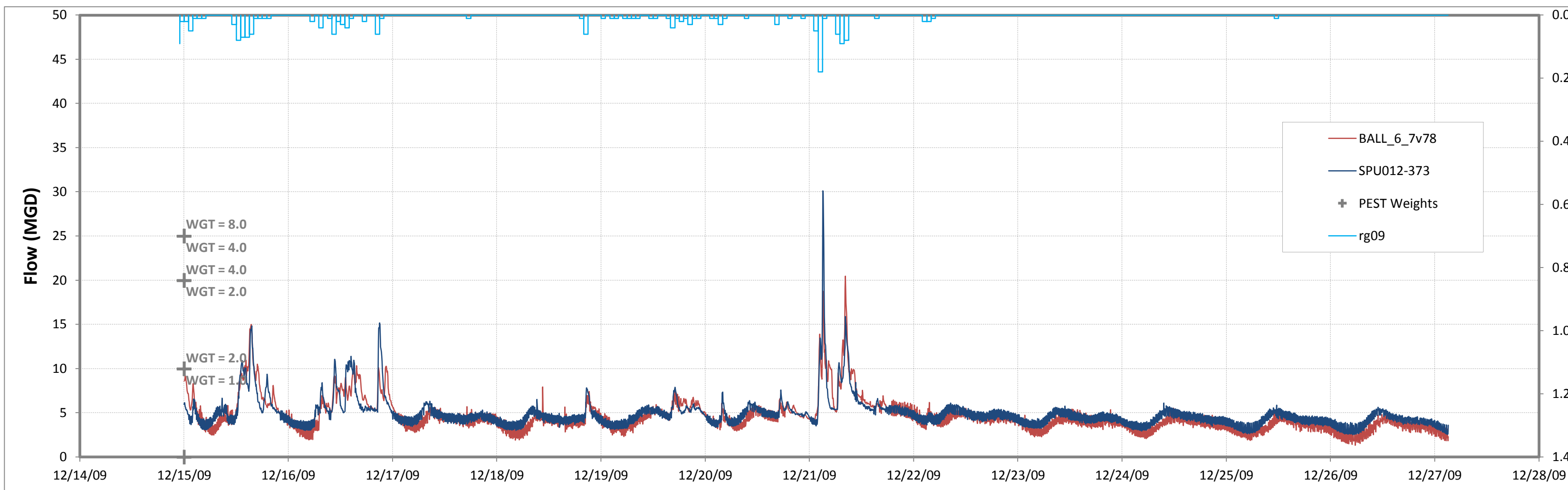
Goodness-of-Fit	
Threshold	0.00
Bias	5.53%
RMSE	1.59
Nash	0.77
Q_{pk} Error	-6.78%
V_{tot} Error	7.22%



Event 05
11/20/09 to 12/02/09

PEST Optimization	
RDII_AREA	8.1 %
B_A_IFLAT	10.0 %
B_M_IFLAT	0.03
UMAX	0.00 mm
LMAX	55.77 mm
CQOF	0.01
CK	9.1 hr
CKIF	538.5 hr
CKBF	2500.0 hr

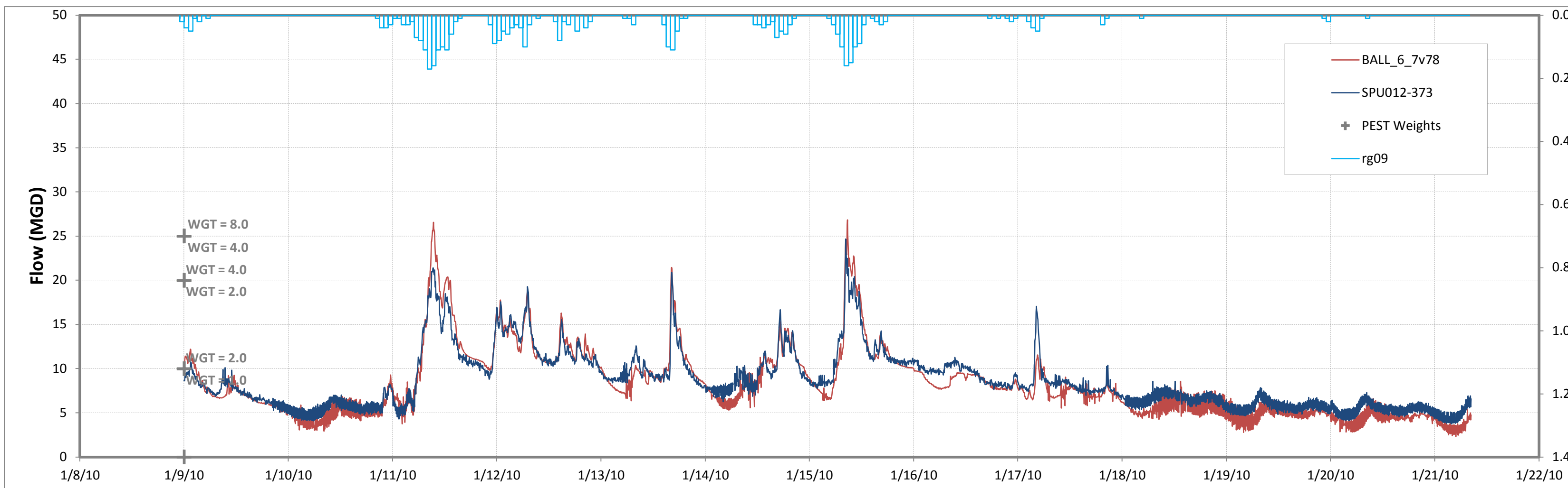
Goodness-of-Fit	
Threshold	0.00
Bias	-1.96%
RMSE	1.20
Nash	0.89
Q_{pk} Error	7.44%
V_{tot} Error	7.58%



Event 06
12/15/09 to 12/27/09

PEST Optimization		
RDII_AREA	8.1	%
B_A_IFLAT	10.0	%
B_M_IFLAT	0.03	
UMAX	0.00	mm
LMAX	55.77	mm
CQOF	0.01	
CK	9.1	hr
CKIF	538.5	hr
CKBF	2500.0	hr

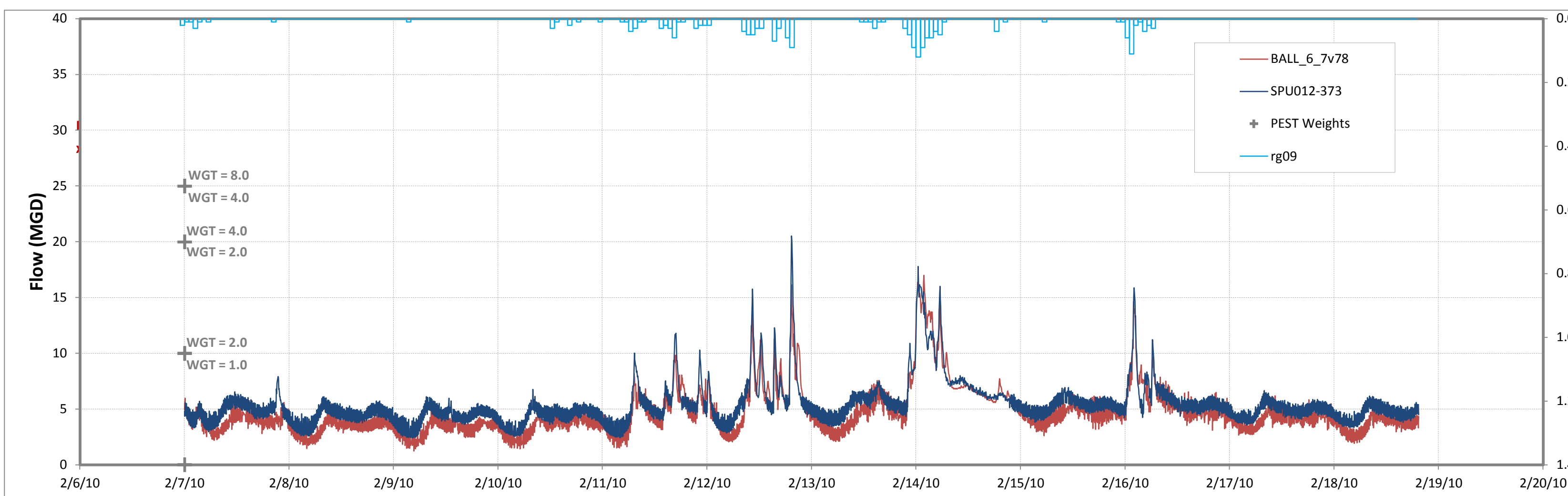
Goodness-of-Fit	
Threshold	0.00
Bias	-5.87%
RMSE	1.14
Nash	0.58
Q_{pk} Error	-32.07%
V_{tot} Error	0.89%



Event 07
01/09/10 to 01/21/10

PEST Optimization		
RDII_AREA	8.1	%
B_A_IFLAT	10.0	%
B_M_IFLAT	0.03	
UMAX	0.00	mm
LMAX	55.77	mm
CQOF	0.01	
CK	9.1	hr
CKIF	538.5	hr
CKBF	2500.0	hr

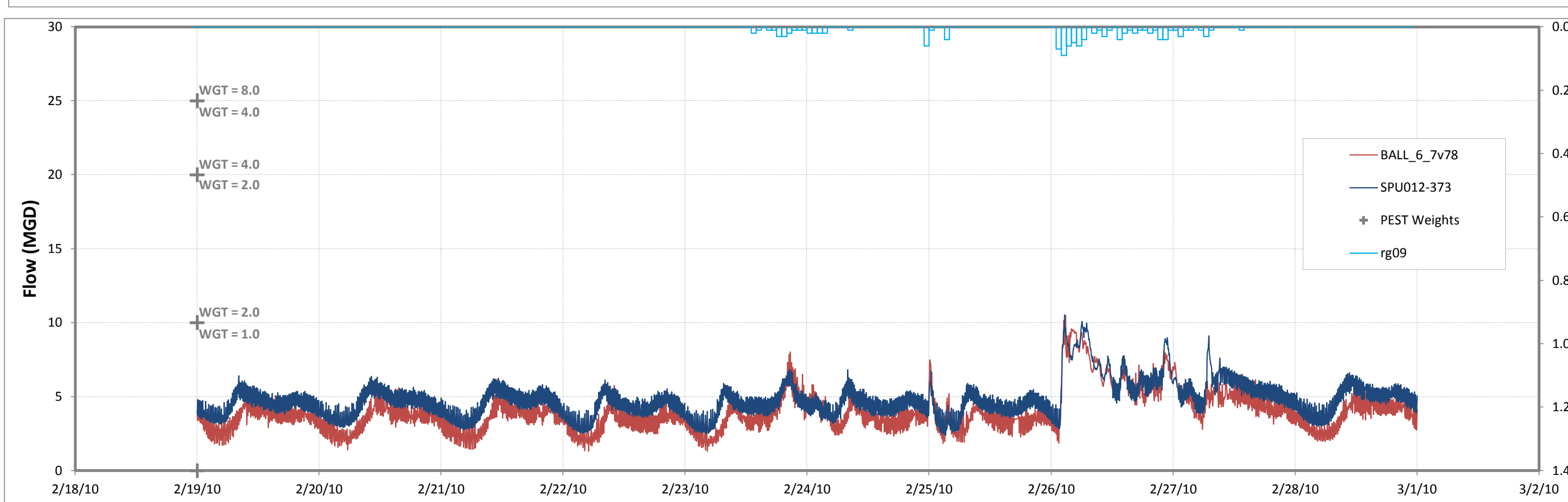
Goodness-of-Fit	
Threshold	0.00
Bias	-4.65%
RMSE	1.28
Nash	0.84
Q_{pk} Error	8.77%
V_{tot} Error	-1.87%



Event 08
02/07/10 to 02/18/10

PEST Optimization		
RDII_AREA	8.1	%
B_A_IFLAT	10.0	%
B_M_IFLAT	0.03	
UMAX	0.00	mm
LMAX	55.77	mm
CQOF	0.01	
CK	9.1	hr
CKIF	538.5	hr
CKBF	2500.0	hr

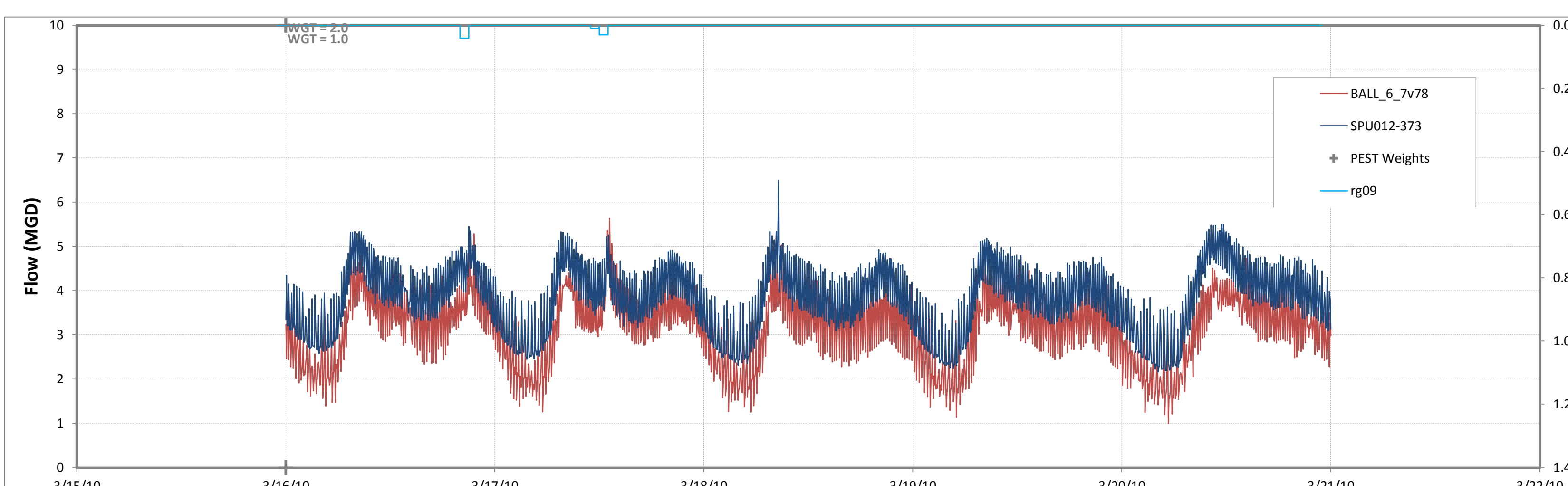
Goodness-of-Fit	
Threshold	0.00
Bias	-12.77%
RMSE	1.26
Nash	0.594
Q_{pk} Error	-17.19%
V_{tot} Error	7.96%



Event 09
02/19/10 to 03/01/10

PEST Optimization		
RDII_AREA	8.1	%
B_A_IFLAT	10.0	%
B_M_IFLAT	0.03	
UMAX	0.00	mm
LMAX	55.77	mm
CQOF	0.01	
CK	9.1	hr
CKIF	538.5	hr
CKBF	2500.0	hr

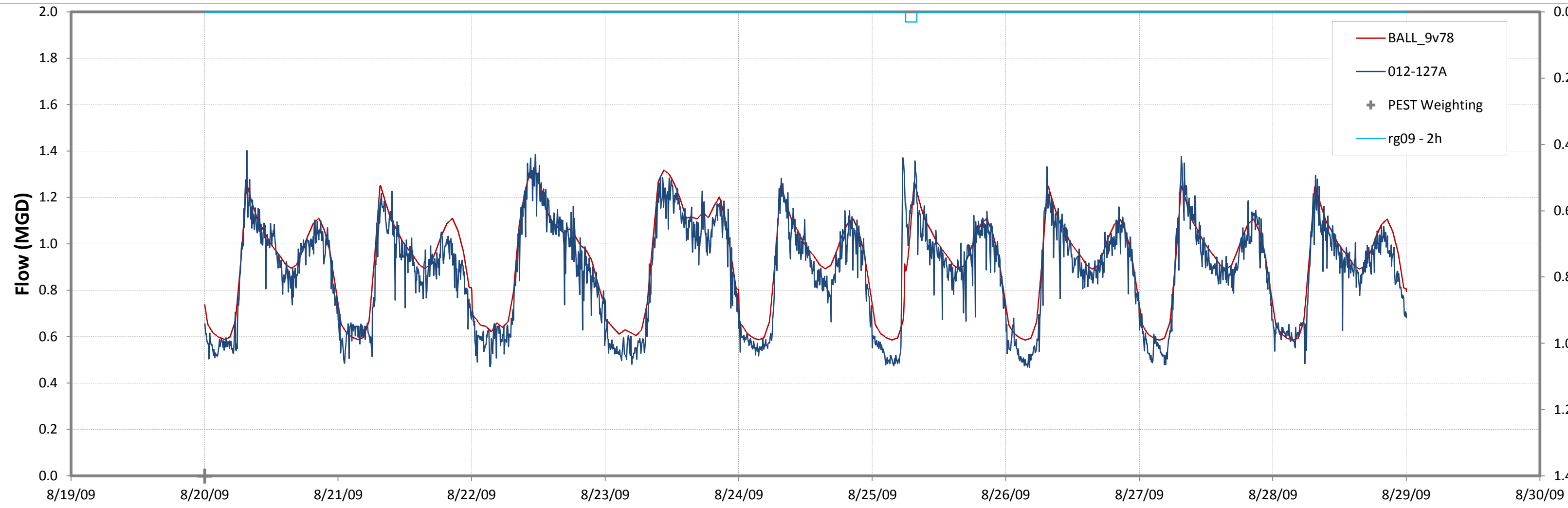
Goodness-of-Fit	
Threshold	0.00
Bias	-16.73%
RMSE	1.18
Nash	0.09
Q_{pk} Error	-0.45%
V_{tot} Error	-8.73%



Event 10
03/16/10 to 03/21/10

PEST Optimization		
RDII_AREA	8.1	%
B_A_IFLAT	10.0	%
B_M_IFLAT	0.03	
UMAX	0.00	mm
LMAX	55.77	mm
CQOF	0.01	
CK	9.1	hr
CKIF	538.5	hr
CKBF	2500.0	hr

Goodness-of-Fit	
Threshold	0.00
Bias	-16.93%
RMSE	1.02
Nash	-0.74
Q_{pk} Error	-13.31%
V_{tot} Error	-13.58%

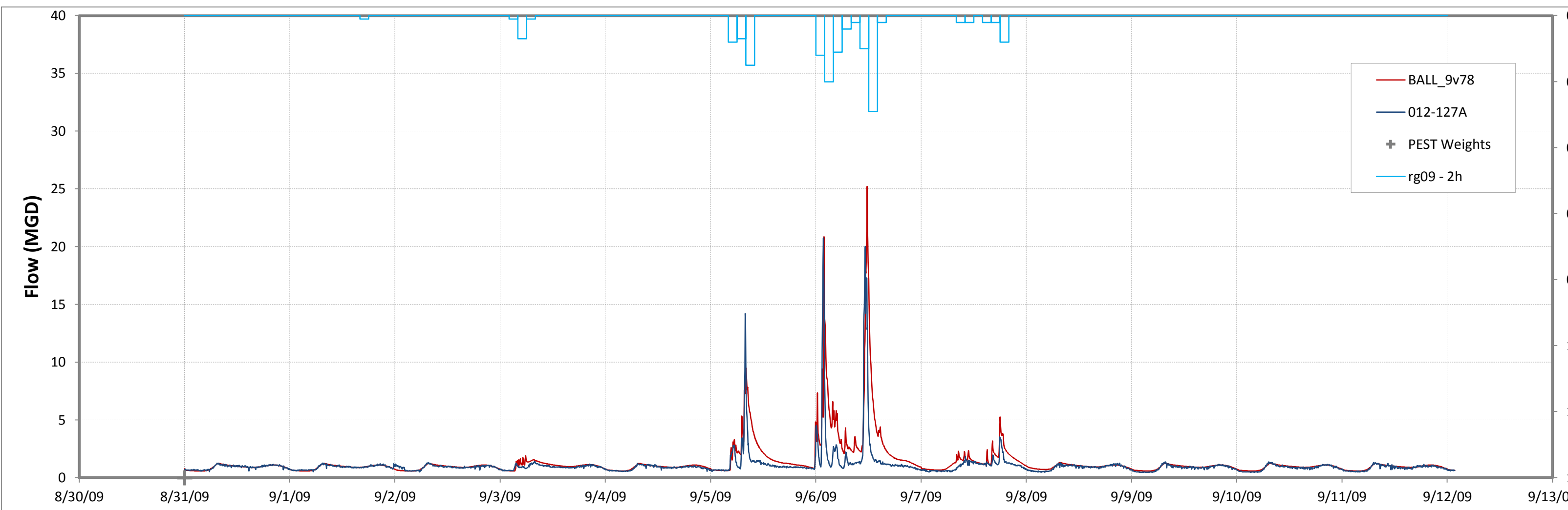


Event 01

08/20/09 to 08/29/09

PEST Optimization	
RDII_AREA	18.1 %
B_A_IFLAT	11.0 %
B_M_IFLAT	99.56
UMAX	0.01 mm
LMAX	52.00 mm
CQOF	0.01
CK	7.9 hr
CKIF	999.7 hr
CKBF	1884.0 hr

Goodness-of-Fit	
Threshold	0.00
Bias	5.06%
RMSE	0.08
Nash	0.86
Q_{pk} Error	-6.03%
V_{tot} Error	76.62%

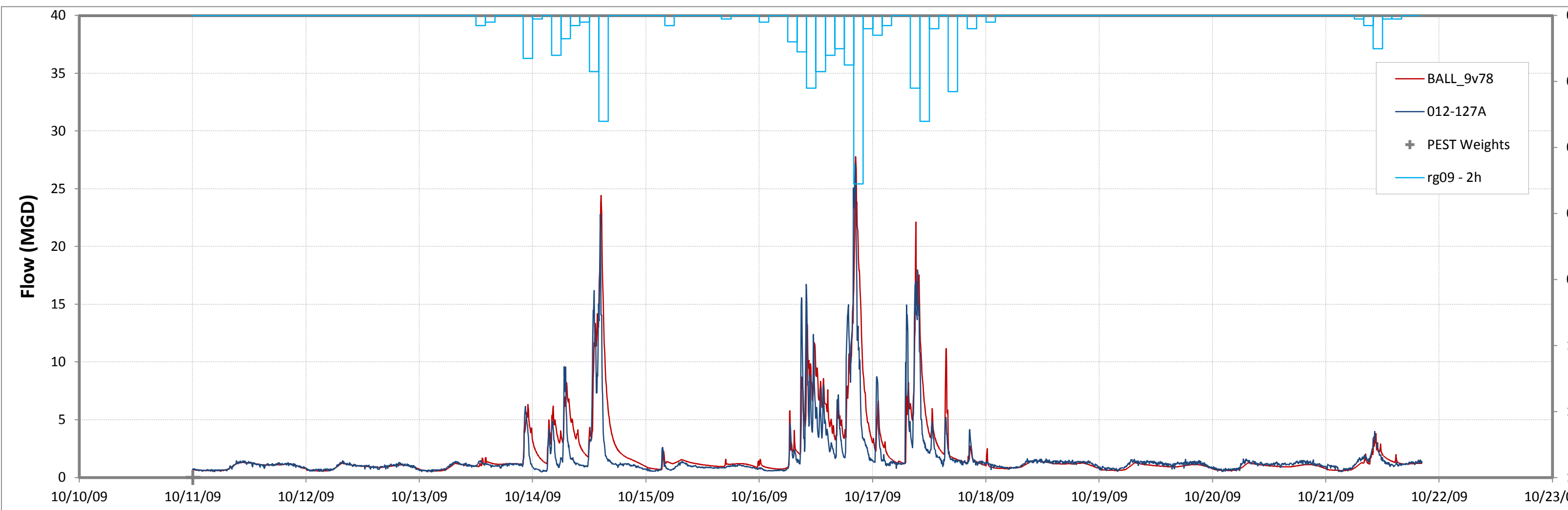


Event 02

08/31/09 to 09/12/09

PEST Optimization	
RDII_AREA	18.1 %
B_A_IFLAT	11.0 %
B_M_IFLAT	99.56
UMAX	0.01 mm
LMAX	52.00 mm
CQOF	0.01
CK	7.9 hr
CKIF	999.7 hr
CKBF	1884.0 hr

Goodness-of-Fit	
Threshold	0.00
Bias	25.06%
RMSE	1.07
Nash	0.27
Q_{pk} Error	21.52%
V_{tot} Error	42.91%

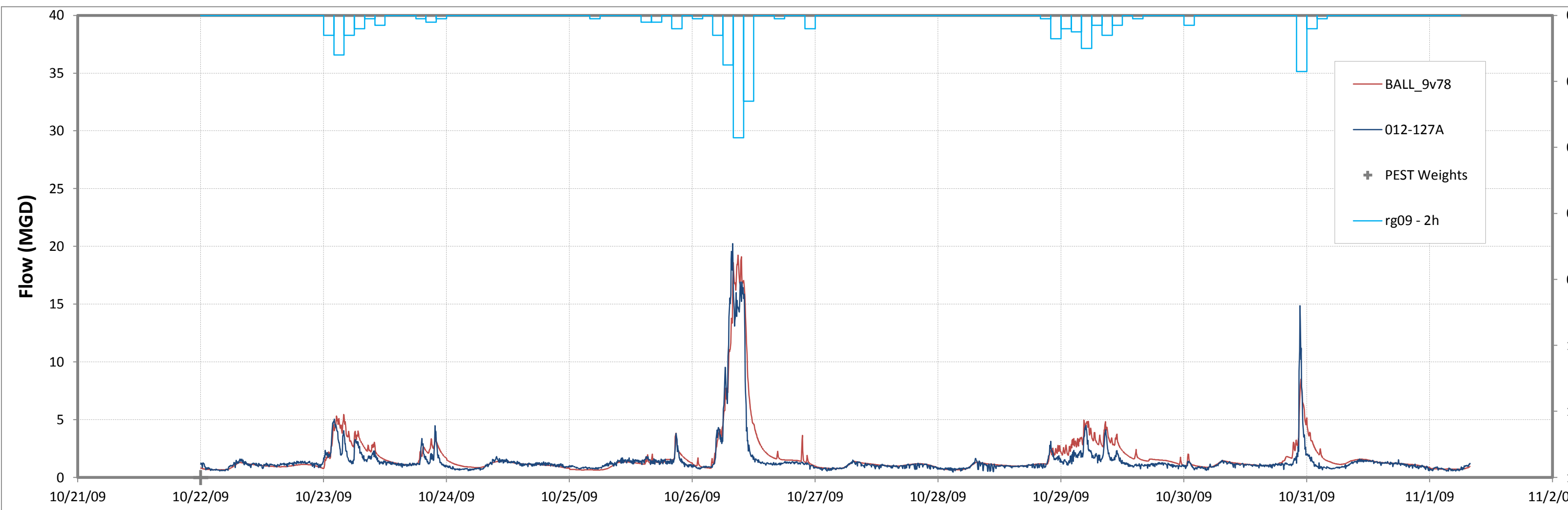


Event 03

10/11/09 to 10/21/09

PEST Optimization	
RDII_AREA	18.1 %
B_A_IFLAT	11.0 %
B_M_IFLAT	99.56
UMAX	0.01 mm
LMAX	52.00 mm
CQOF	0.01
CK	7.9 hr
CKIF	999.7 hr
CKBF	1884.0 hr

Goodness-of-Fit	
Threshold	0.00
Bias	16.59%
RMSE	1.46
Nash	0.67
Q_{pk} Error	2.42%
V_{tot} Error	22.78%

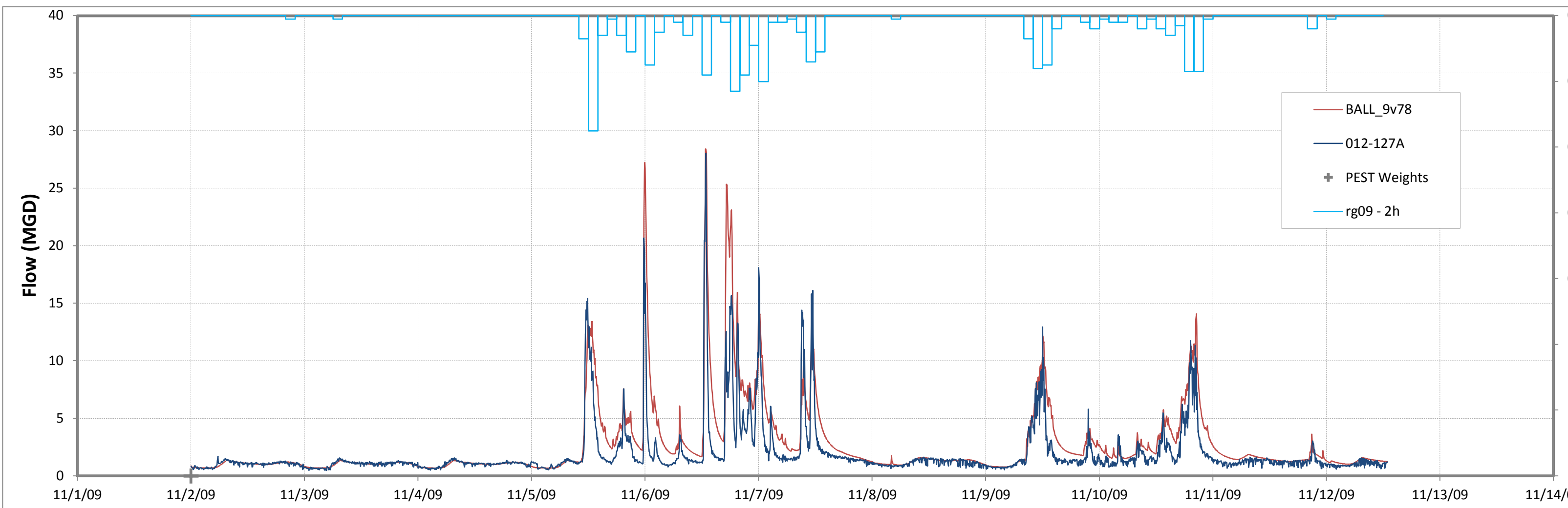


Event 04

10/22/09 to 11/01/09

PEST Optimization	
RDII_AREA	18.1 %
B_A_IFLAT	11.0 %
B_M_IFLAT	99.56
UMAX	0.01 mm
LMAX	52.00 mm
CQOF	0.01
CK	7.9 hr
CKIF	999.7 hr
CKBF	1884.0 hr

Goodness-of-Fit	
Threshold	0.00
Bias	16.71%
RMSE	0.81
Nash	0.81
Q_{pk} Error	-4.96%
V_{tot} Error	14.96%

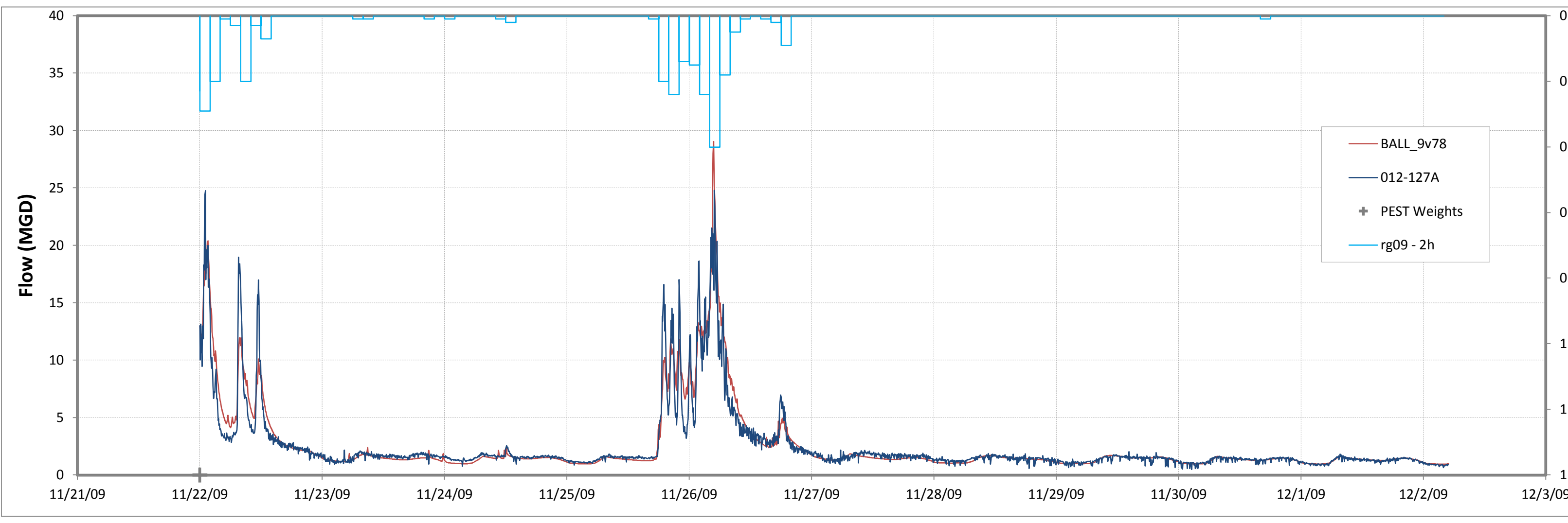


Event 05

11/02/09 to 11/12/09

PEST Optimization	
RDII_AREA	18.1 %
B_A_IFLAT	11.0 %
B_M_IFLAT	99.56
UMAX	0.01 mm
LMAX	52.00 mm
CQOF	0.01
CK	7.9 hr
CKIF	999.7 hr
CKBF	1884.0 hr

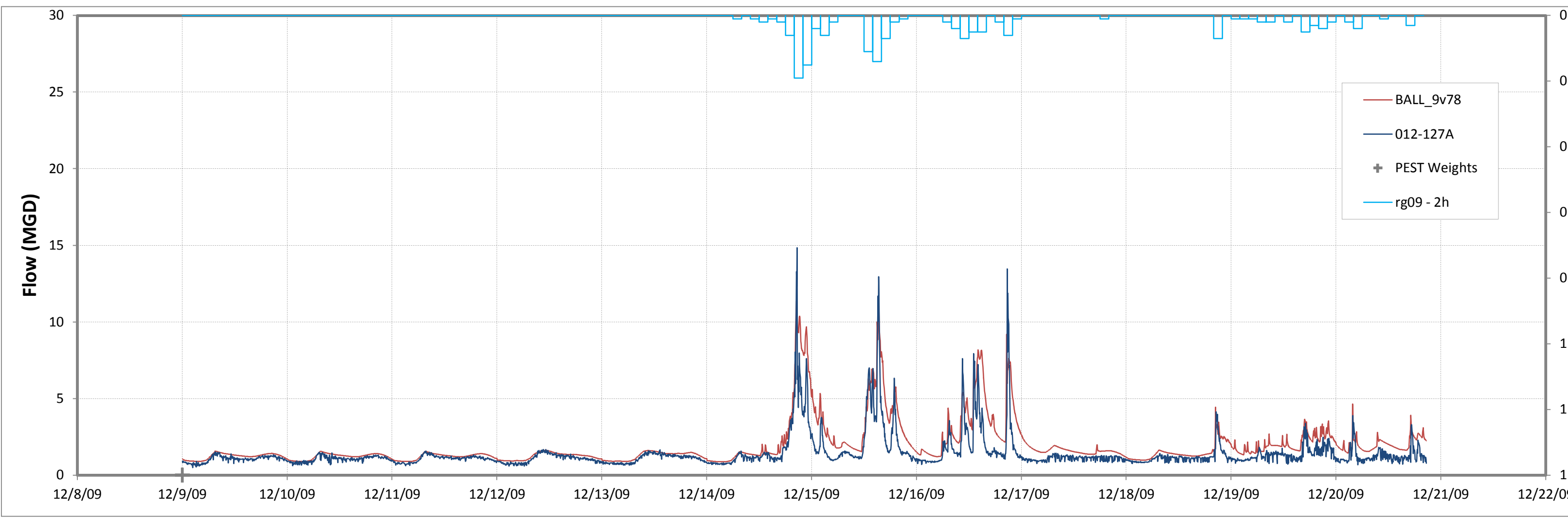
Goodness-of-Fit	
Threshold	0.00
Bias	38.44%
RMSE	1.81
Nash	0.45
Q_{pk} Error	1.39%
V_{tot} Error	63.04%



Event 06
11/22/09 to 12/02/09

PEST Optimization	
RDII_AREA	18.1 %
B_A_IFLAT	11.0 %
B_M_IFLAT	99.56
UMAX	0.01 mm
LMAX	52.00 mm
CQOF	0.01
CK	7.9 hr
CKIF	999.7 hr
CKBF	1884.0 hr

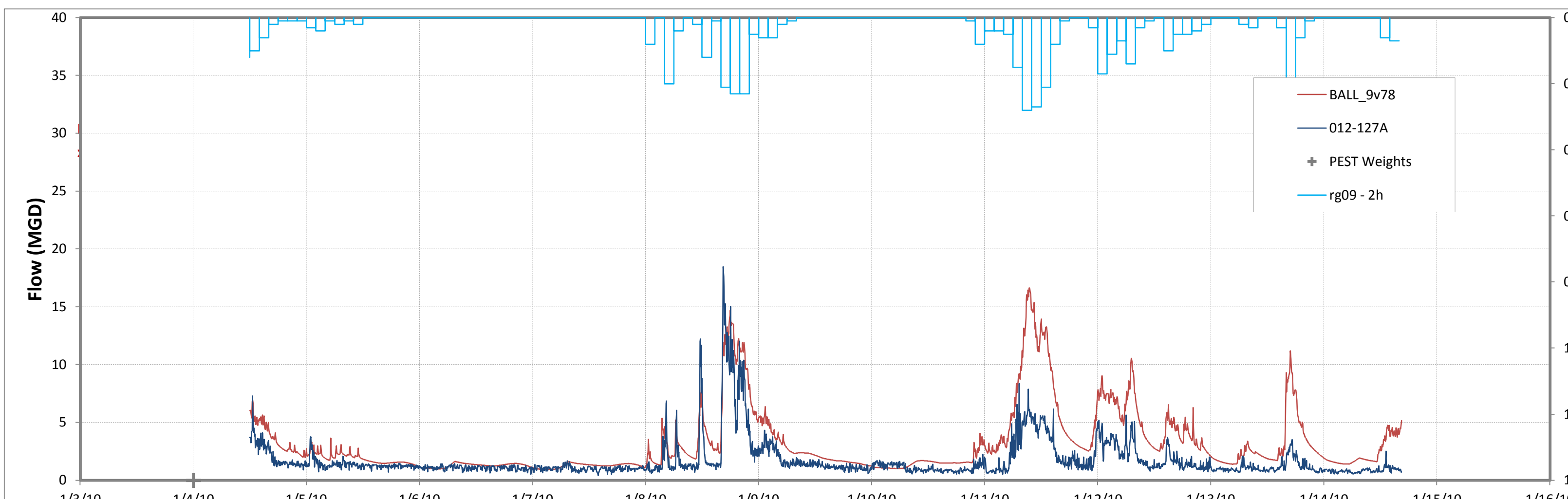
Goodness-of-Fit	
Threshold	0.00
Bias	1.63%
RMSE	1.01
Nash	0.89
Q_{pk} Error	17.21%
V_{tot} Error	4.85%



Event 07
12/09/09 to 12/20/09

PEST Optimization	
RDII_AREA	18.1 %
B_A_IFLAT	11.0 %
B_M_IFLAT	99.56
UMAX	0.01 mm
LMAX	52.00 mm
CQOF	0.01
CK	7.9 hr
CKIF	999.7 hr
CKBF	1884.0 hr

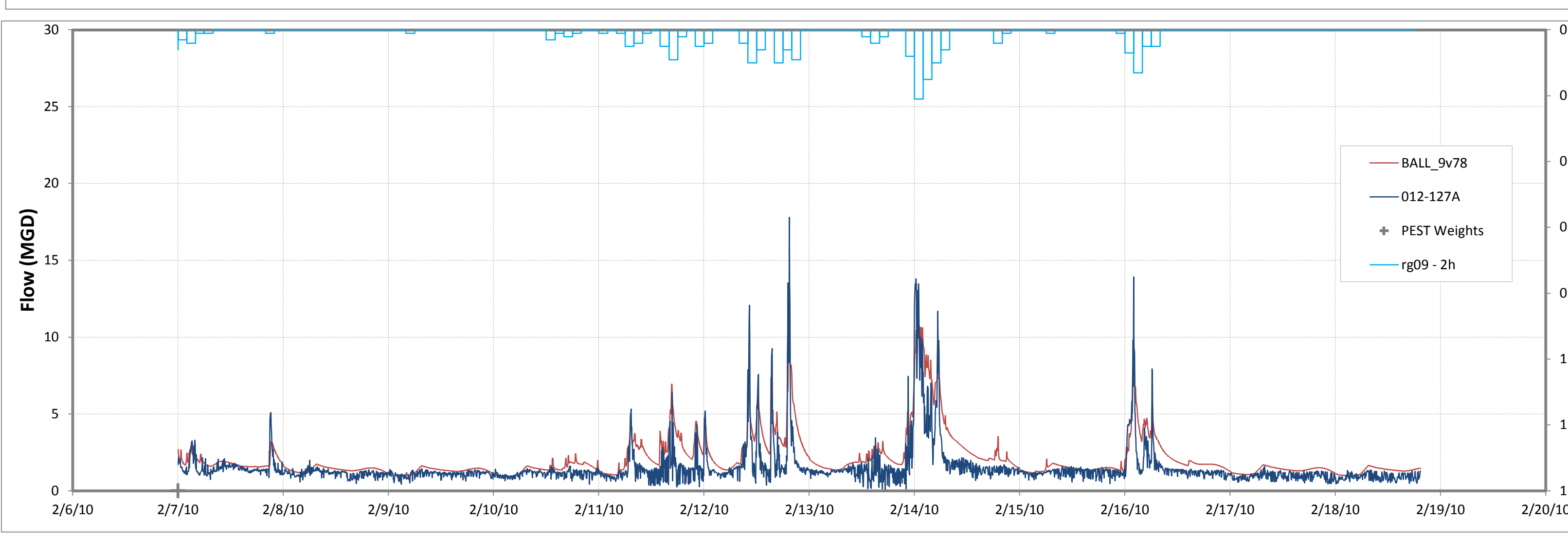
Goodness-of-Fit	
Threshold	0.00
Bias	36.41%
RMSE	0.90
Nash	0.36
Q_{pk} Error	-23.57%
V_{tot} Error	39.97%



Event 08
01/04/10 to 01/14/10

PEST Optimization	
RDII_AREA	18.1 %
B_A_IFLAT	11.0 %
B_M_IFLAT	99.56
UMAX	0.01 mm
LMAX	52.00 mm
CQOF	0.01
CK	7.9 hr
CKIF	999.7 hr
CKBF	1884.0 hr

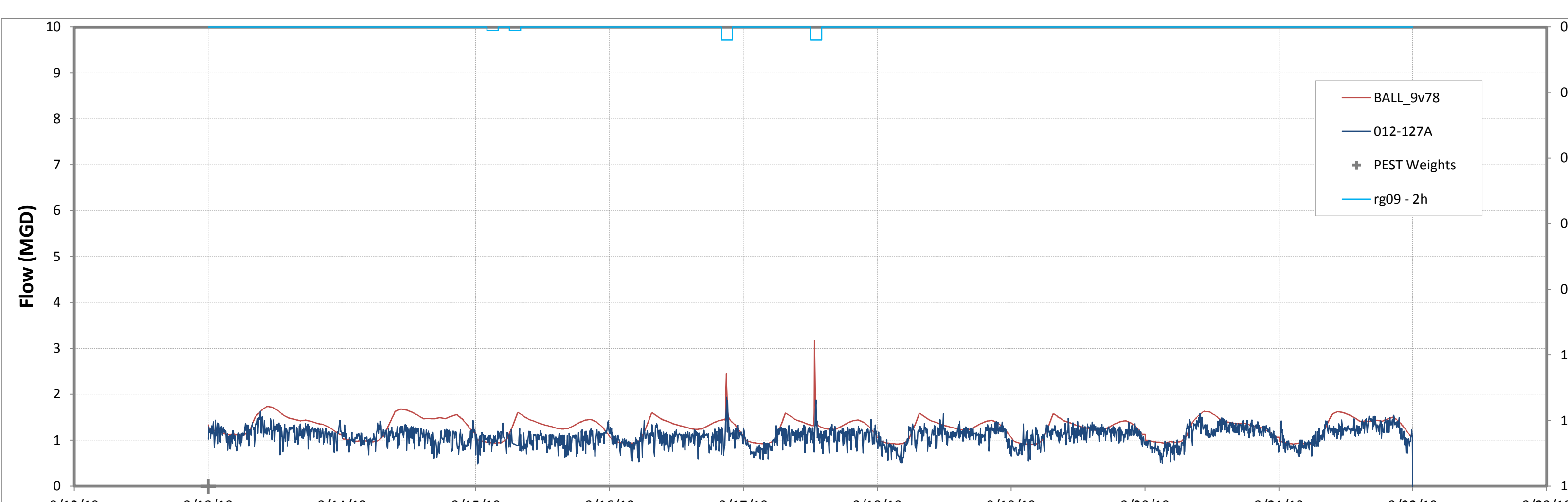
Goodness-of-Fit	
Threshold	0.00
Bias	81.99%
RMSE	2.24
Nash	-0.749
Q_{pk} Error	-9.89%
V_{tot} Error	97.01%



Event 09
02/07/10 to 02/18/10

PEST Optimization	
RDII_AREA	18.1 %
B_A_IFLAT	11.0 %
B_M_IFLAT	99.56
UMAX	0.01 mm
LMAX	52.00 mm
CQOF	0.01
CK	7.9 hr
CKIF	999.7 hr
CKBF	1884.0 hr

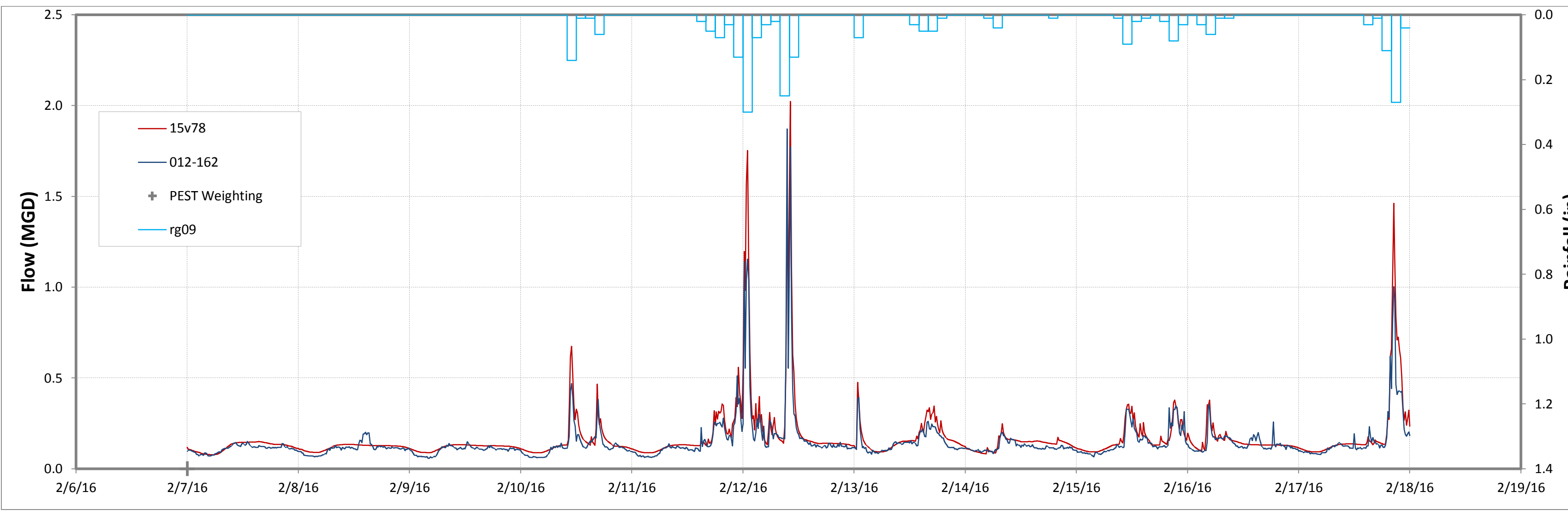
Goodness-of-Fit	
Threshold	0.00
Bias	38.64%
RMSE	1.02
Nash	0.46
Q_{pk} Error	-34.19%
V_{tot} Error	48.15%



Event 10
03/13/10 to 03/22/10

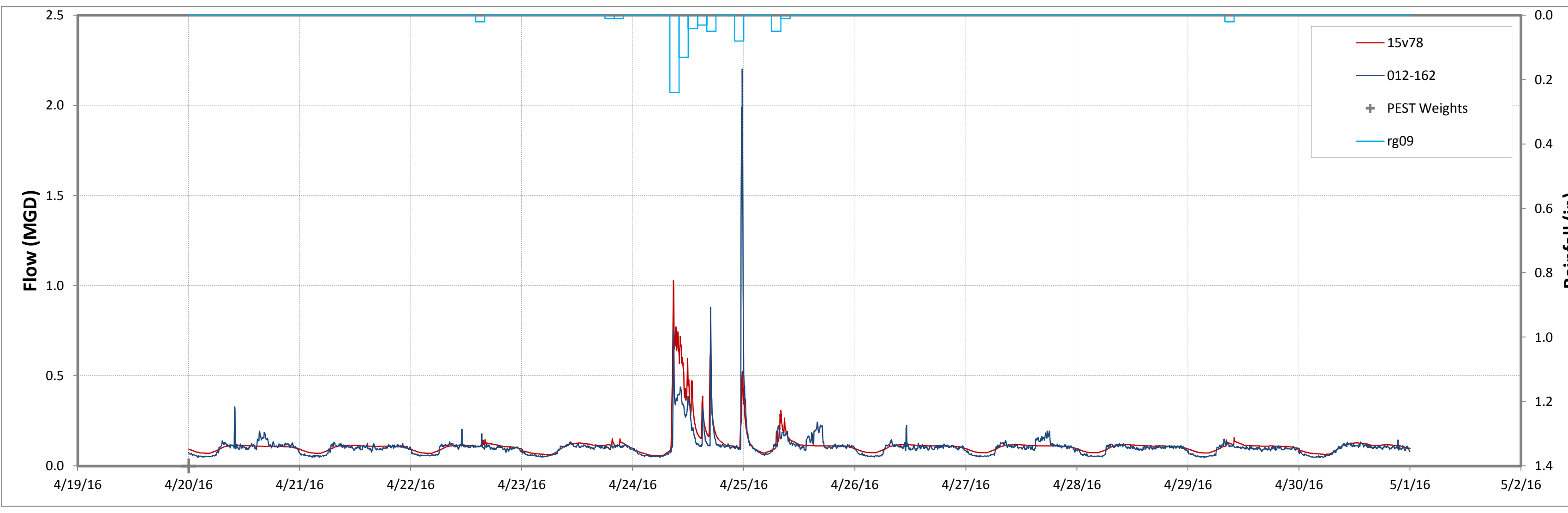
PEST Optimization	
RDII_AREA	18.1 %
B_A_IFLAT	11.0 %
B_M_IFLAT	99.56
UMAX	0.01 mm
LMAX	52.00 mm
CQOF	0.01
CK	7.9 hr
CKIF	999.7 hr
CKBF	1884.0 hr

Goodness-of-Fit	
Threshold	0.00
Bias	17.45%
RMSE	0.29
Nash	-1.21
Q_{pk} Error	64.10%
V_{tot} Error	20.22%



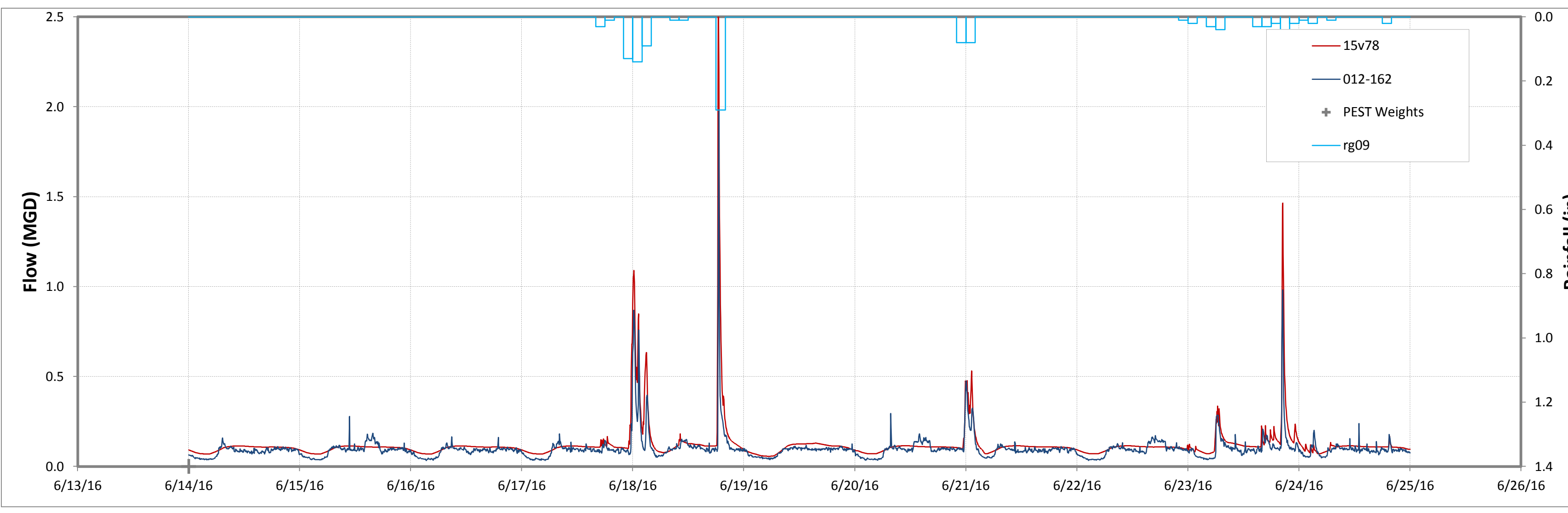
Event 01
02/07/16 to 02/18/16

PEST Optimization	
RDII_AREA	6.5 %
B_A_IFLAT	9.5 %
B_M_IFLAT	3.22
UMAX	66.54 mm
LMAX	97.26 mm
CQOF	0.78
CK	10.6 hr
CKIF	382.2 hr
CKBF	1000.0 hr
Goodness-of-Fit	
Threshold	0.00
Bias	15.20%
RMSE	0.06
Nash	0.79
Q_{pk} Error	8.12%
V_{tot} Error	17.68%



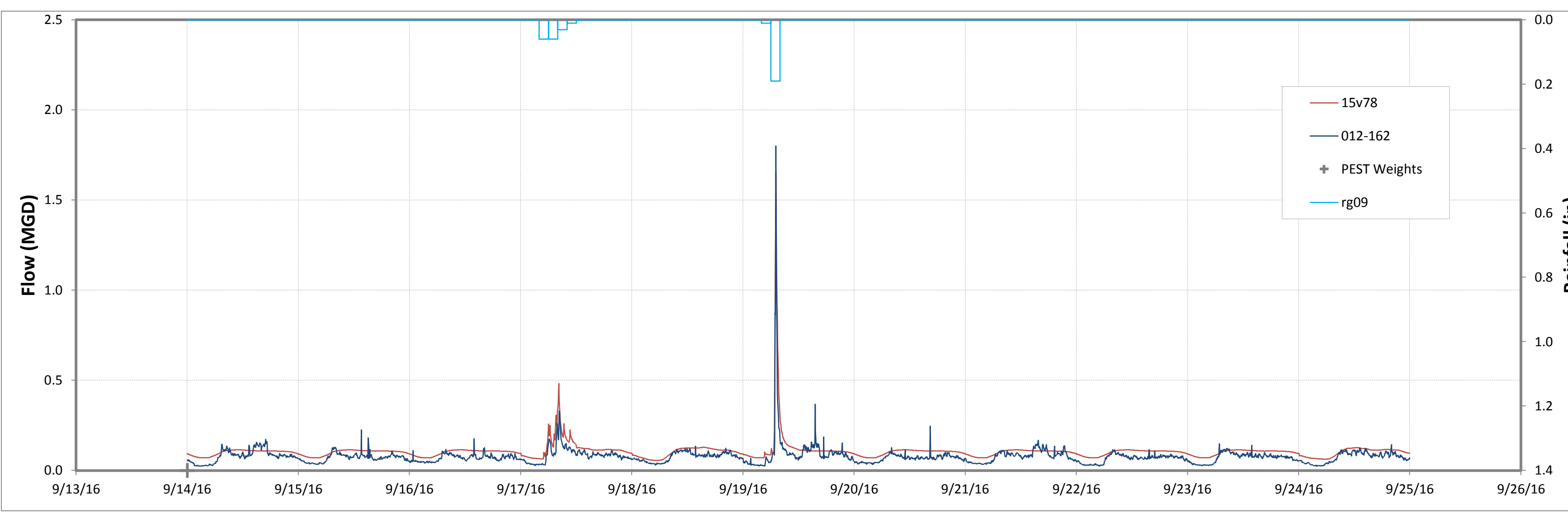
Event 02
04/20/16 to 05/01/16

PEST Optimization	
RDII_AREA	6.5 %
B_A_IFLAT	9.5 %
B_M_IFLAT	3.22
UMAX	66.54 mm
LMAX	97.26 mm
CQOF	0.78
CK	10.6 hr
CKIF	382.2 hr
CKBF	1000.0 hr
Goodness-of-Fit	
Threshold	0.00
Bias	7.70%
RMSE	0.07
Nash	0.36
Q_{pk} Error	-53.30%
V_{tot} Error	7.42%



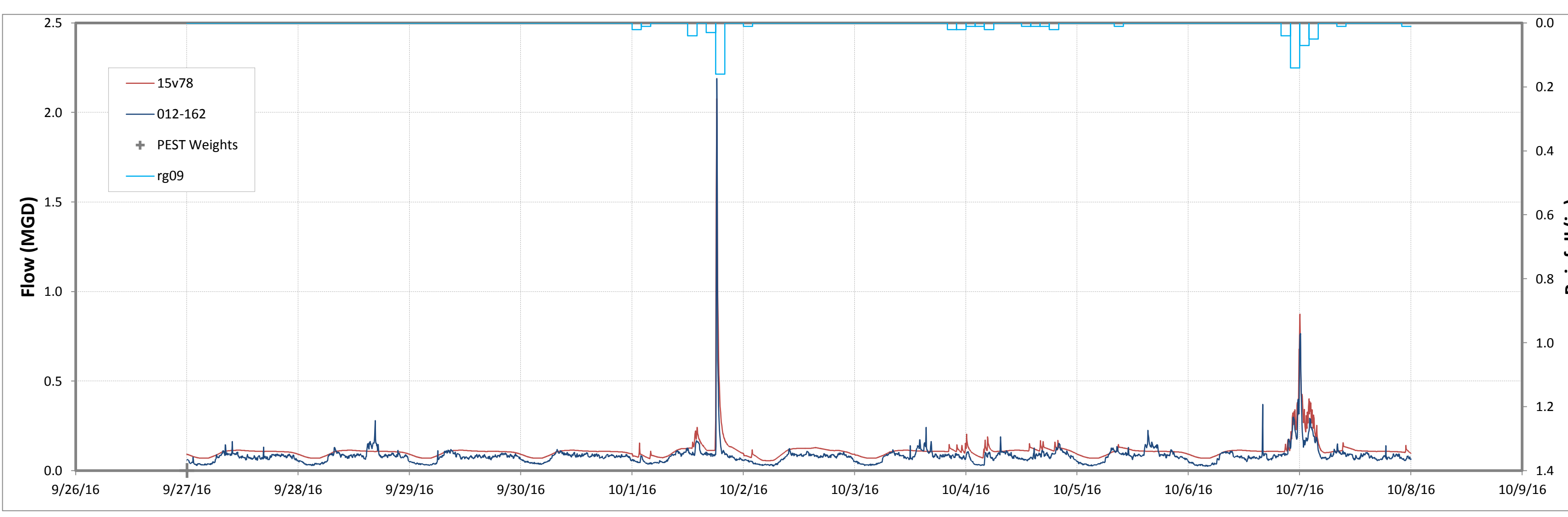
Event 03
06/14/16 to 06/25/16

PEST Optimization	
RDII_AREA	6.5 %
B_A_IFLAT	9.5 %
B_M_IFLAT	3.22
UMAX	66.54 mm
LMAX	97.26 mm
CQOF	0.78
CK	10.6 hr
CKIF	382.2 hr
CKBF	1000.0 hr
Goodness-of-Fit	
Threshold	0.00
Bias	23.70%
RMSE	0.07
Nash	0.27
Q_{pk} Error	35.36%
V_{tot} Error	38.98%



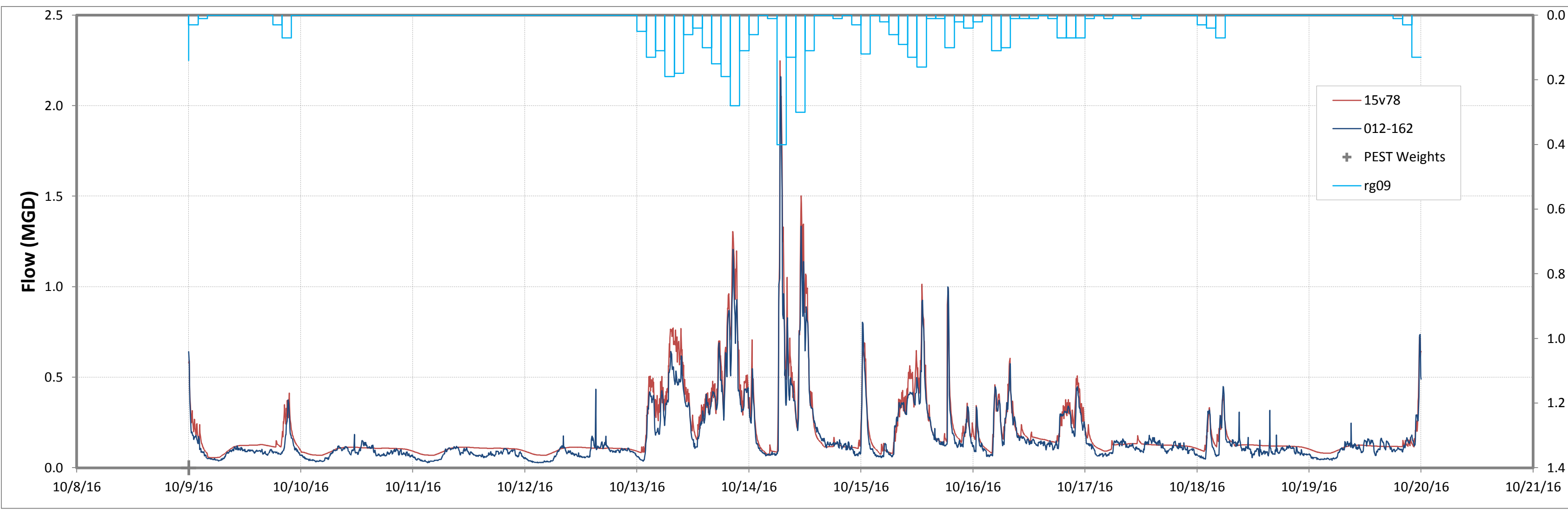
Event 04
09/14/16 to 09/25/16

PEST Optimization	
RDII_AREA	6.5 %
B_A_IFLAT	9.5 %
B_M_IFLAT	3.22
UMAX	66.54 mm
LMAX	97.26 mm
CQOF	0.78
CK	10.6 hr
CKIF	382.2 hr
CKBF	1000.0 hr
Goodness-of-Fit	
Threshold	0.00
Bias	36.66%
RMSE	0.04
Nash	0.57
Q_{pk} Error	-8.32%
V_{tot} Error	31.24%



Event 05
09/27/16 to 10/08/16

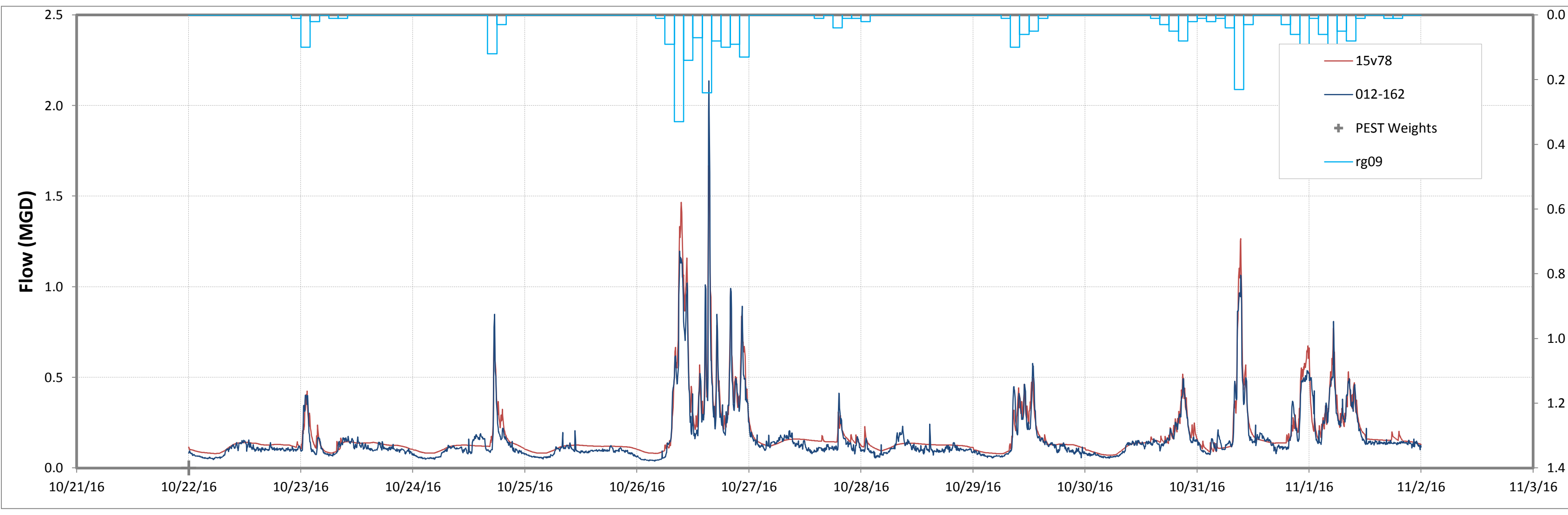
PEST Optimization	
RDII_AREA	6.5 %
B_A_IFLAT	9.5 %
B_M_IFLAT	3.22
UMAX	66.54 mm
LMAX	97.26 mm
CQOF	0.78
CK	10.6 hr
CKIF	382.2 hr
CKBF	1000.0 hr
Goodness-of-Fit	
Threshold	0.00
Bias	35.62%
RMSE	0.04
Nash	0.60
Q_{pk} Error	-5.11%
V_{tot} Error	44.79%



Event 06
10/09/16 to 10/20/16

PEST Optimization	
RDII_AREA	6.5 %
B_A_IFLAT	9.5 %
B_M_IFLAT	3.22
UMAX	66.54 mm
LMAX	97.26 mm
CQOF	0.78
CK	10.6 hr
CKIF	382.2 hr
CKBF	1000.0 hr

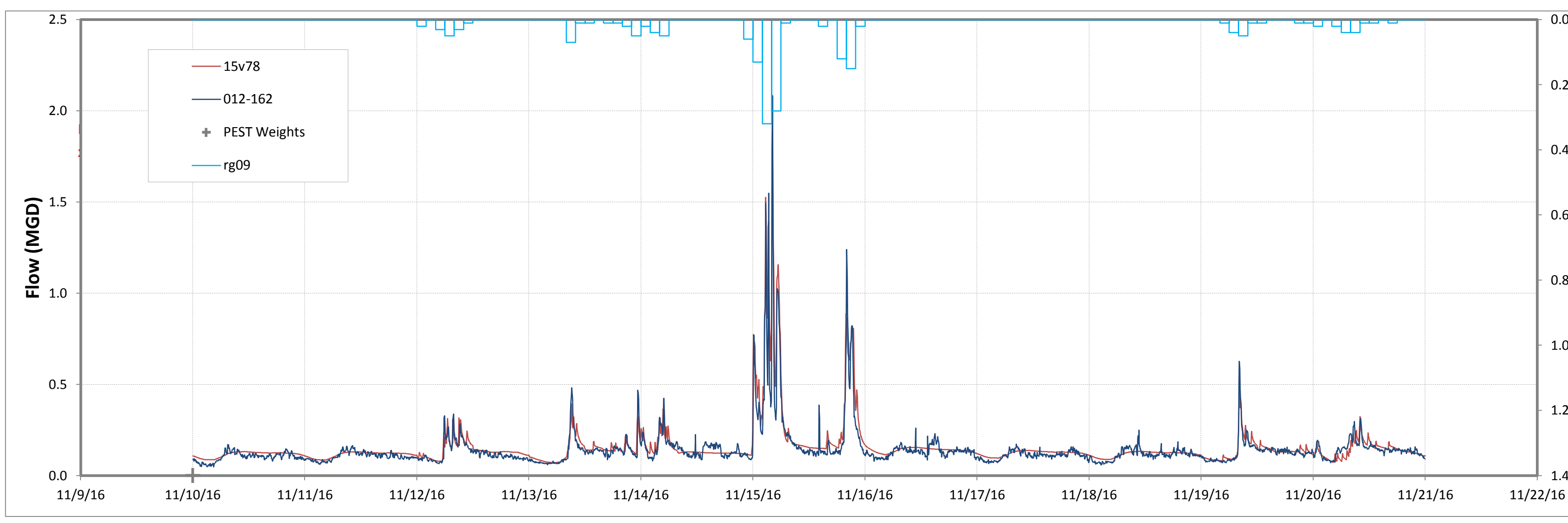
Goodness-of-Fit	
Threshold	0.00
Bias	17.57%
RMSE	0.06
Nash	0.89
Q_{pk} Error	4.04%
V_{tot} Error	16.32%



Event 07
10/22/16 to 11/02/16

PEST Optimization	
RDII_AREA	6.5 %
B_A_IFLAT	9.5 %
B_M_IFLAT	3.22
UMAX	66.54 mm
LMAX	97.26 mm
CQOF	0.78
CK	10.6 hr
CKIF	382.2 hr
CKBF	1000.0 hr

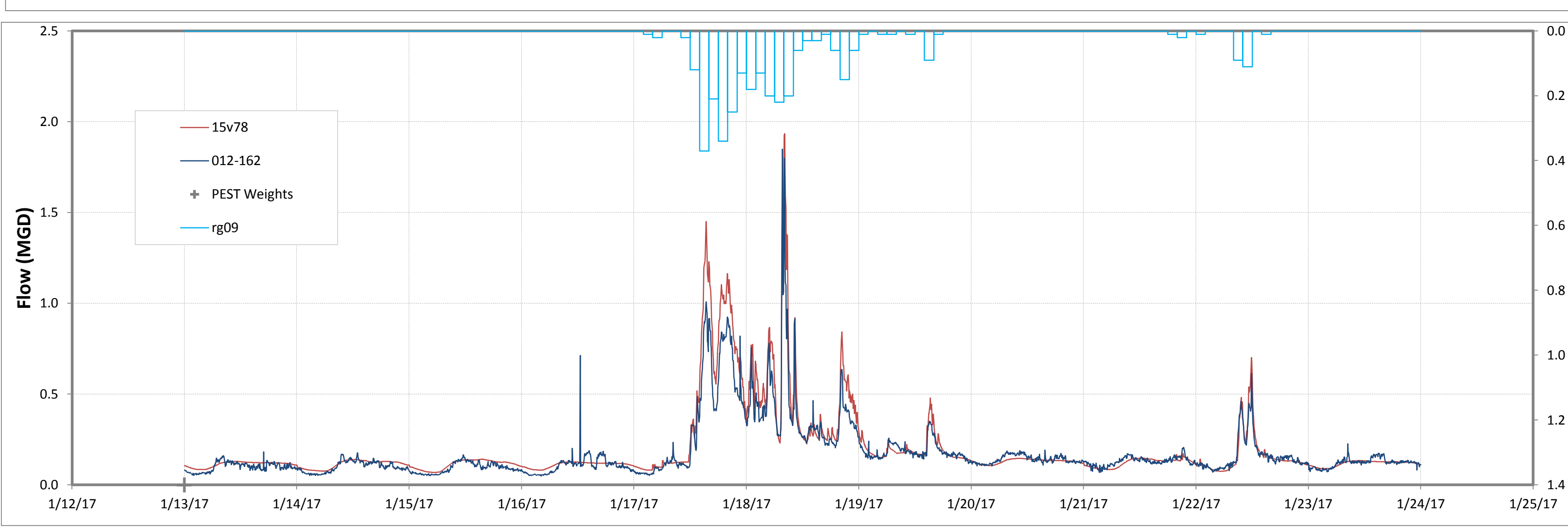
Goodness-of-Fit	
Threshold	0.00
Bias	11.85%
RMSE	0.05
Nash	0.90
Q_{pk} Error	-21.89%
V_{tot} Error	12.14%



Event 08
11/10/16 to 11/21/16

PEST Optimization	
RDII_AREA	6.5 %
B_A_IFLAT	9.5 %
B_M_IFLAT	3.22
UMAX	66.54 mm
LMAX	97.26 mm
CQOF	0.78
CK	10.6 hr
CKIF	382.2 hr
CKBF	1000.0 hr

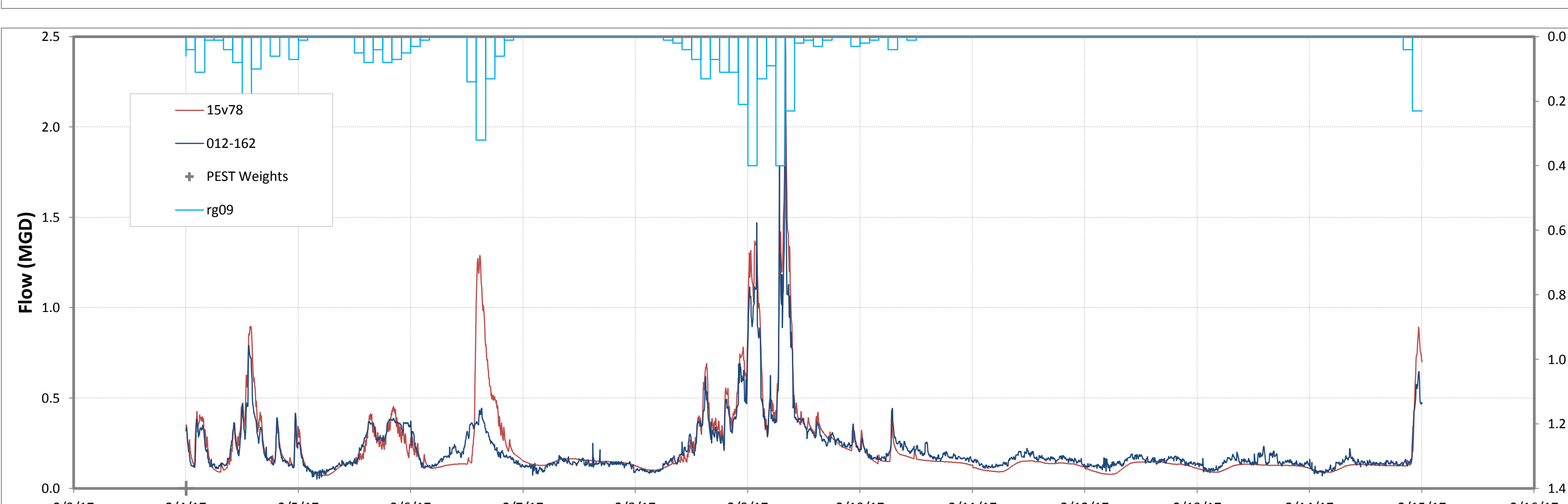
Goodness-of-Fit	
Threshold	0.00
Bias	8.63%
RMSE	0.04
Nash	0.863
Q_{pk} Error	-12.35%
V_{tot} Error	13.12%



Event 09
01/13/17 to 01/24/17

PEST Optimization	
RDII_AREA	6.5 %
B_A_IFLAT	9.5 %
B_M_IFLAT	3.22
UMAX	66.54 mm
LMAX	97.26 mm
CQOF	0.78
CK	10.6 hr
CKIF	382.2 hr
CKBF	1000.0 hr

Goodness-of-Fit	
Threshold	0.00
Bias	11.67%
RMSE	0.06
Nash	0.84
Q_{pk} Error	4.63%
V_{tot} Error	22.22%



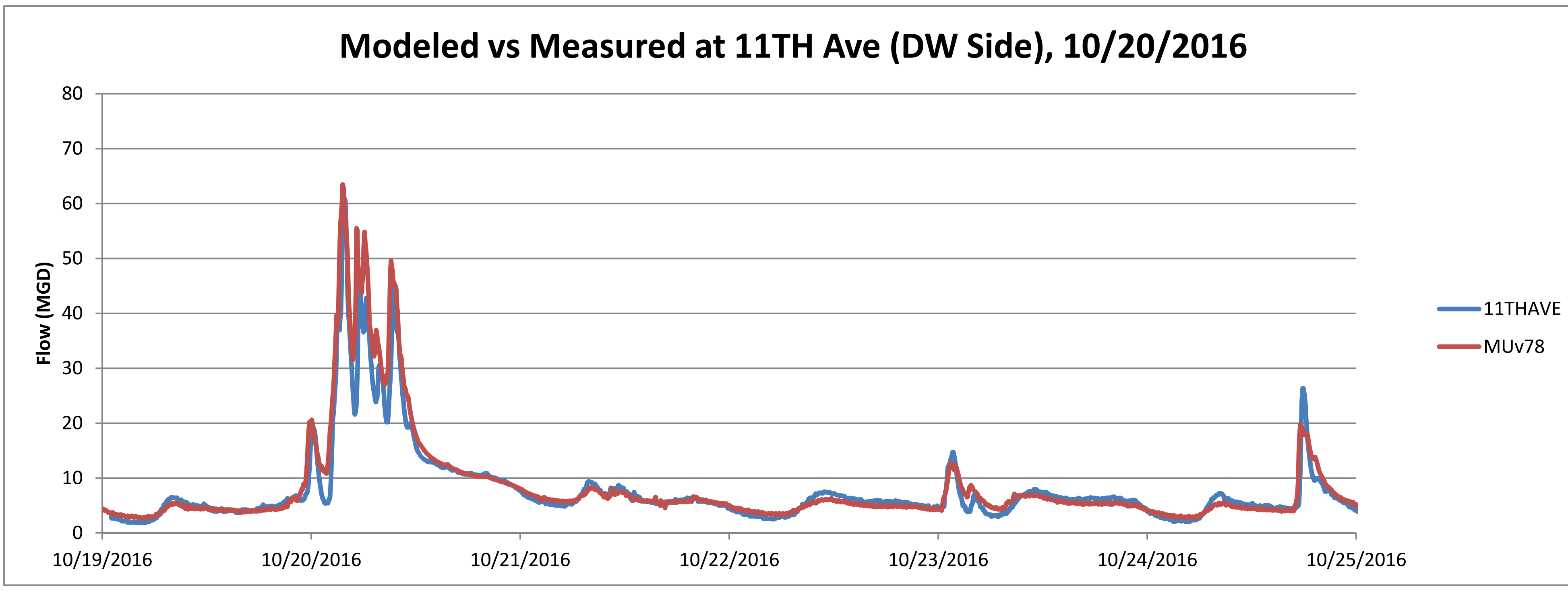
Event 10
02/04/17 to 02/15/17

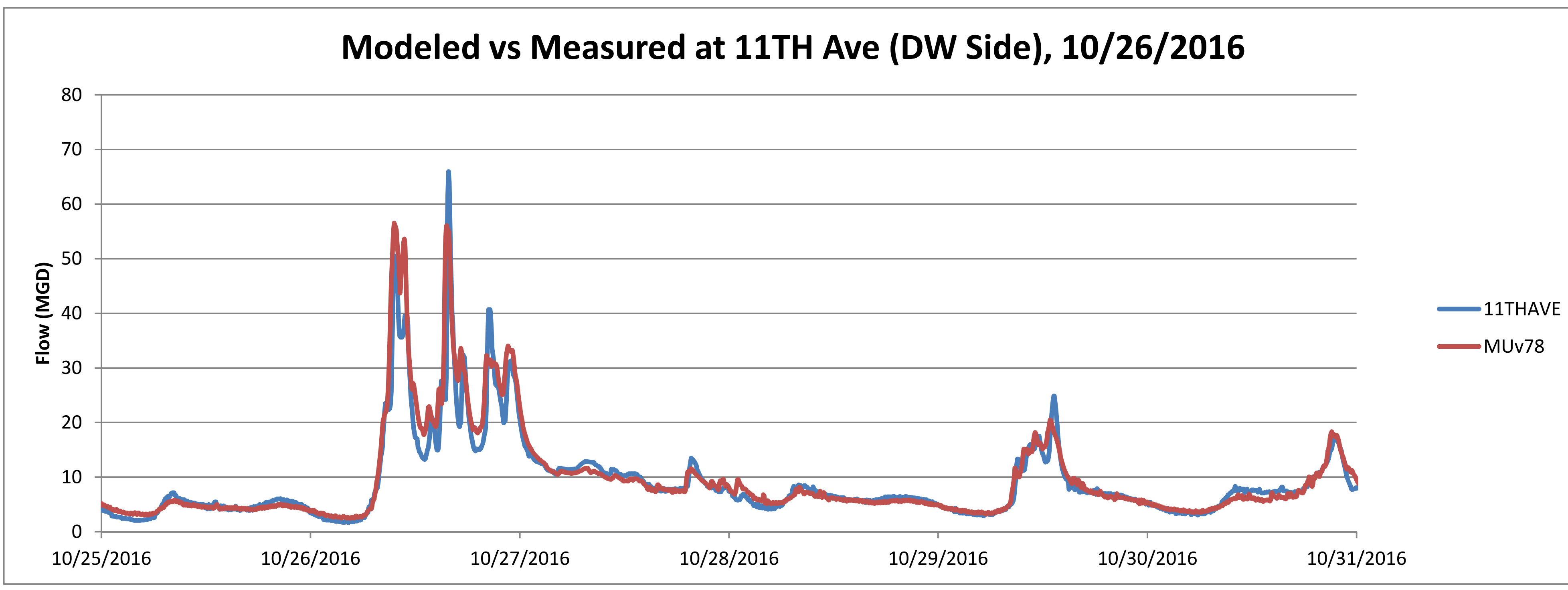
PEST Optimization	
RDII_AREA	6.5 %
B_A_IFLAT	9.5 %
B_M_IFLAT	3.22
UMAX	66.54 mm
LMAX	97.26 mm
CQOF	0.78
CK	10.6 hr
CKIF	382.2 hr
CKBF	1000.0 hr

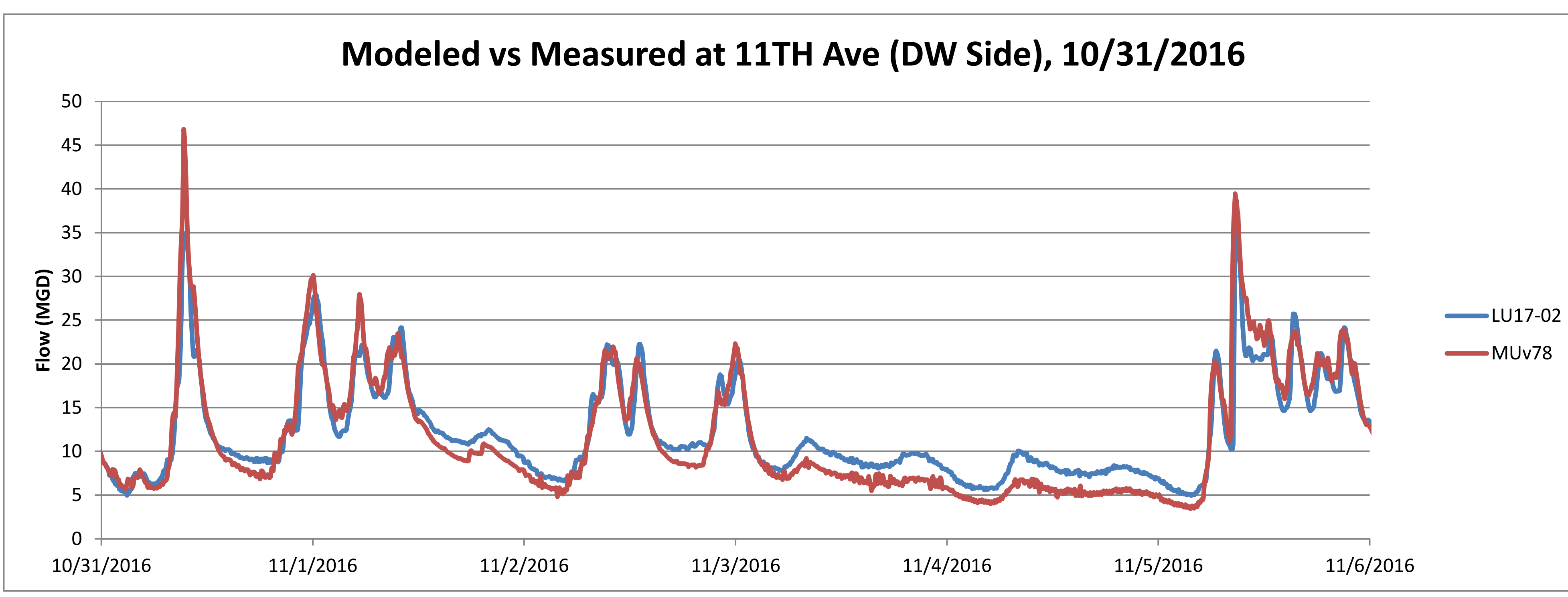
Goodness-of-Fit	
Threshold	0.00
Bias	3.03%
RMSE	0.09
Nash	0.70
Q_{pk} Error	-20.18%
V_{tot} Error	8.39%

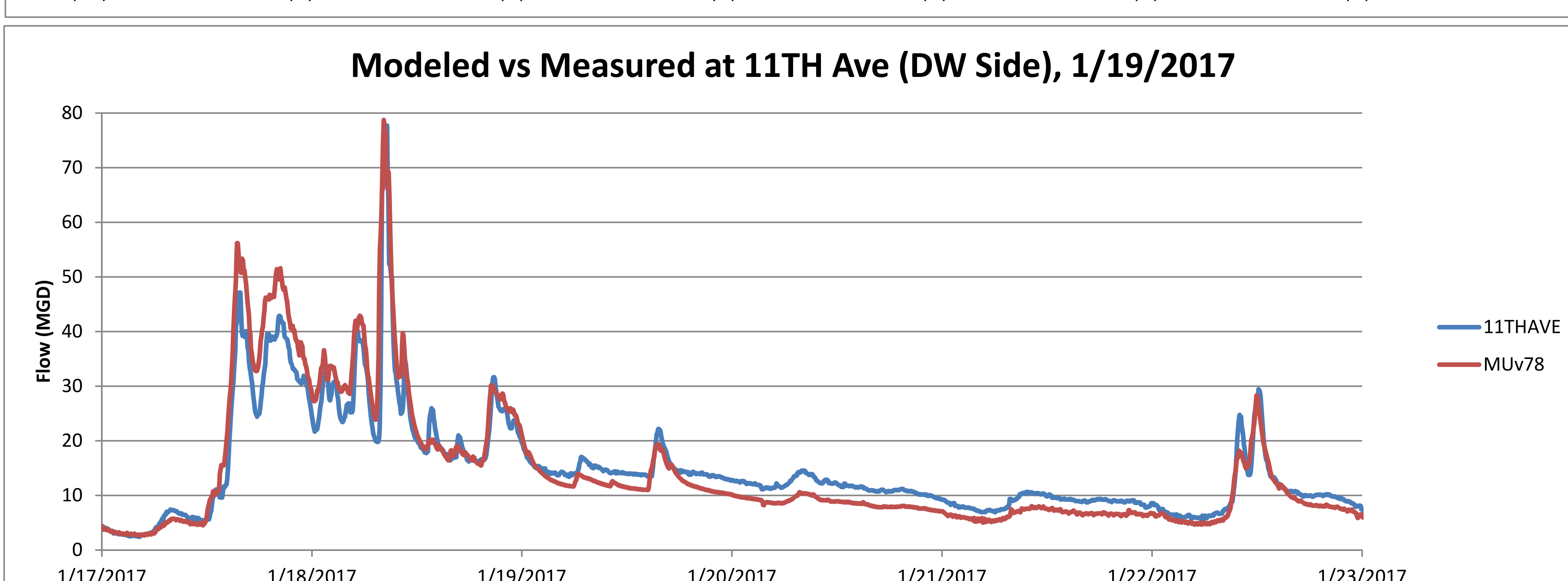
Appendix B: Hydraulic Verification Graphs

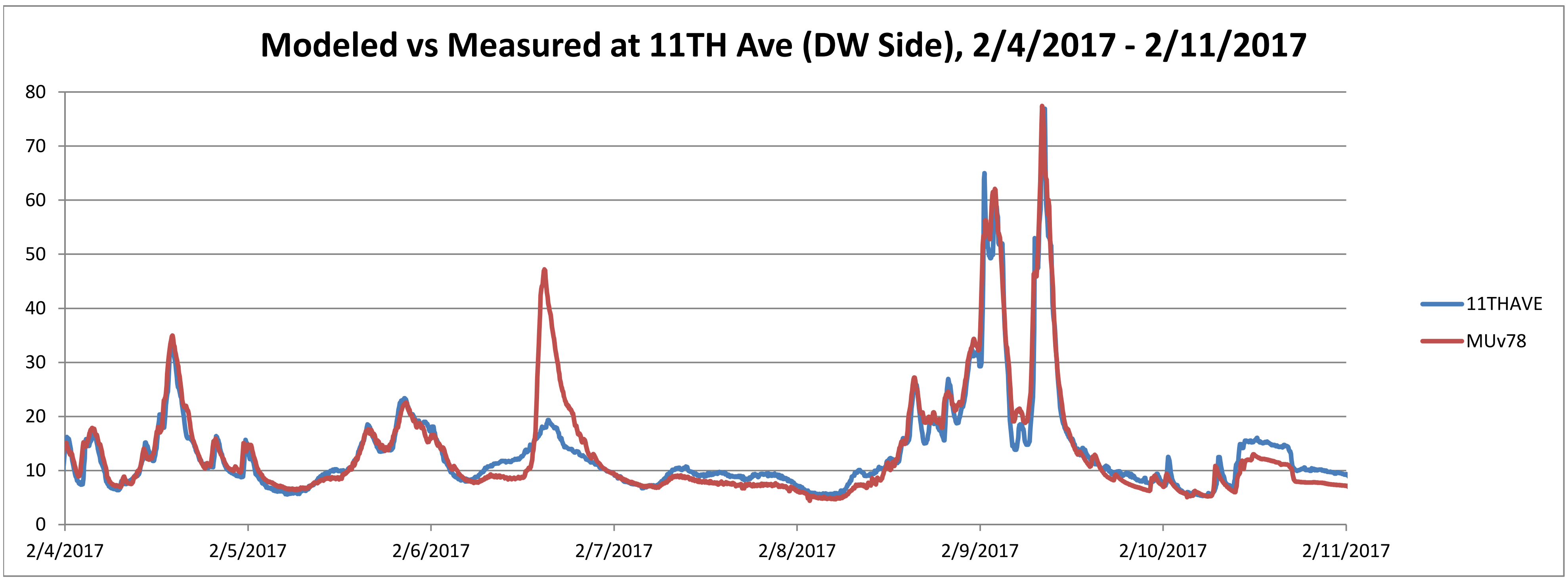
11th Ave NW CSO Model Development and Calibration Report

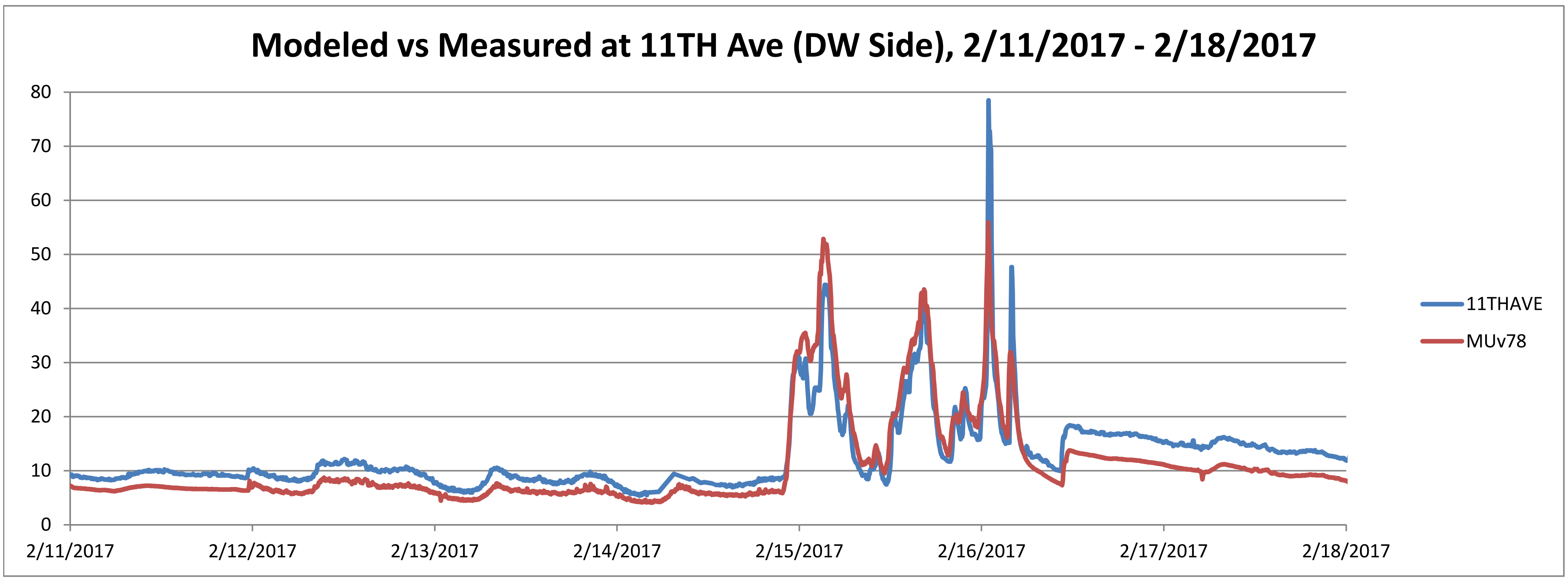

Modeled vs Measured at 11TH Ave (DW Side), 10/20/2016
Flow (MGD)
0
10
20
30
40
50
60
70
80
10/19/2016
10/20/2016
10/21/2016
10/22/2016
10/23/2016
10/24/2016
10/25/2016
11THAVE
MUv78

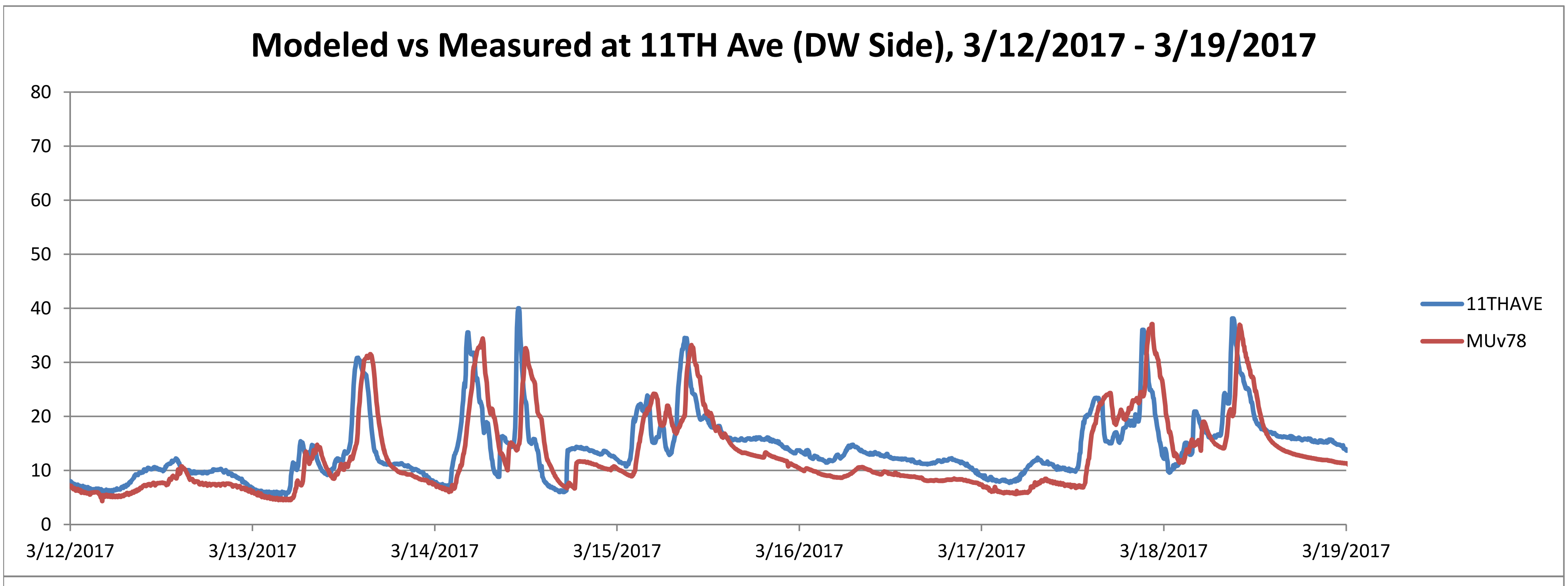

Modeled vs Measured at 11TH Ave (DW Side), 10/26/2016
Flow (MGD)
0
10
20
30
40
50
60
70
80
10/25/2016
10/26/2016
10/27/2016
10/28/2016
10/29/2016
10/30/2016
10/31/2016
11THAVE
MUv78

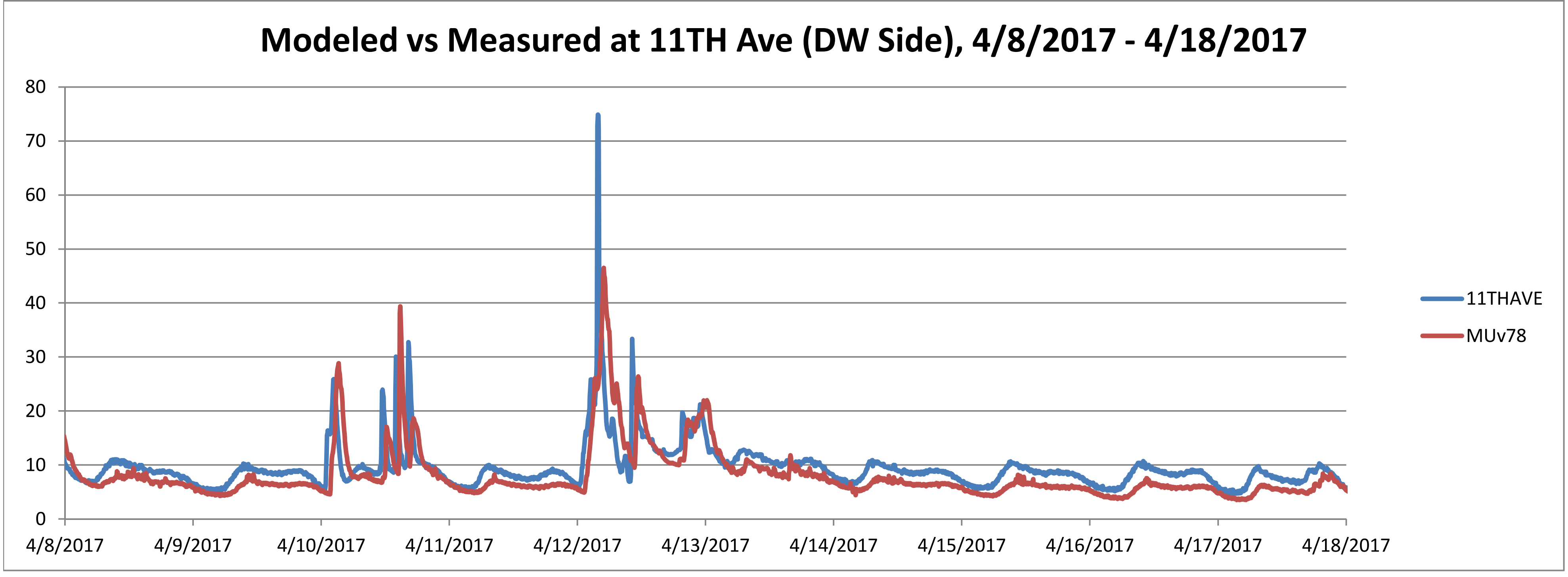

Modeled vs Measured at 11TH Ave (DW Side), 10/31/2016
Flow (MGD)
0
5
10
15
20
25
30
35
40
45
50
10/31/2016
11/1/2016
11/2/2016
11/3/2016
11/4/2016
11/5/2016
11/6/2016
LU17-02
MUv78


Modeled vs Measured at 11TH Ave (DW Side), 1/19/2017
Flow (MGD)
0
10
20
30
40
50
60
70
80
1/17/2017
1/18/2017
1/19/2017
1/20/2017
1/21/2017
1/22/2017
1/23/2017
11THAVE
MUv78


Modeled vs Measured at 11TH Ave (DW Side), 2/4/2017 - 2/11/2017
0
10
20
30
40
50
60
70
80
2/4/2017
2/5/2017
2/6/2017
2/7/2017
2/8/2017
2/9/2017
2/10/2017
2/11/2017
11THAVE
MUv78


Modeled vs Measured at 11TH Ave (DW Side), 2/11/2017 - 2/18/2017
0
10
20
30
40
50
60
70
80
2/11/2017
2/12/2017
2/13/2017
2/14/2017
2/15/2017
2/16/2017
2/17/2017
2/18/2017
11THAVE
MUv78


Modeled vs Measured at 11TH Ave (DW Side), 3/12/2017 - 3/19/2017
0
10
20
30
40
50
60
70
80
3/12/2017
3/13/2017
3/14/2017
3/15/2017
3/16/2017
3/17/2017
3/18/2017
3/19/2017
11THAVE
MUv78


Modeled vs Measured at 11TH Ave (DW Side), 4/8/2017 - 4/18/2017
0
10
20
30
40
50
60
70
80
4/8/2017
4/9/2017
4/10/2017
4/11/2017
4/12/2017
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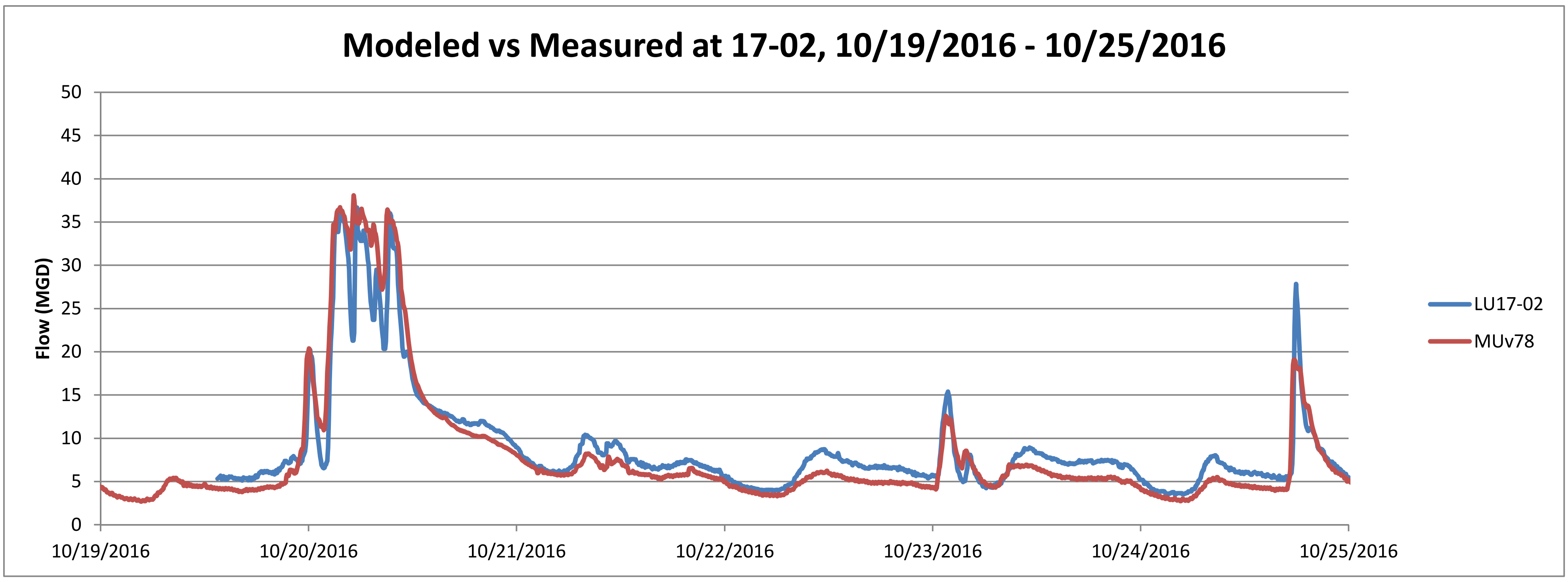
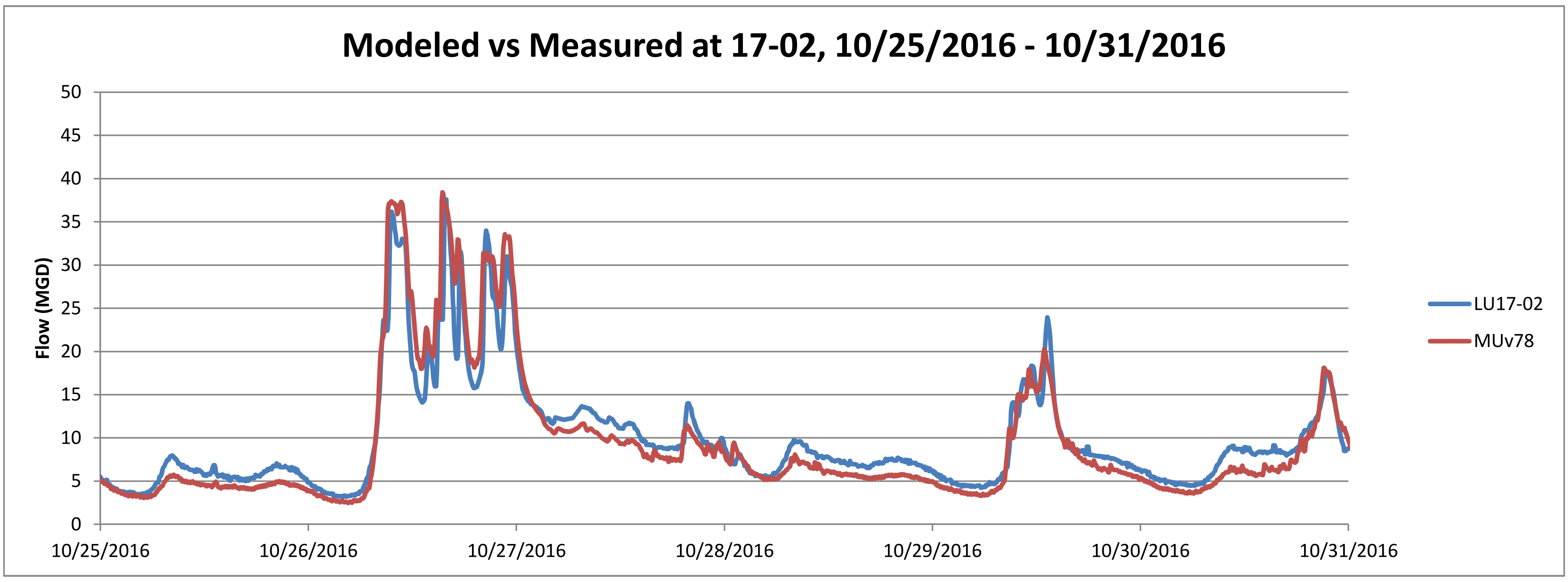
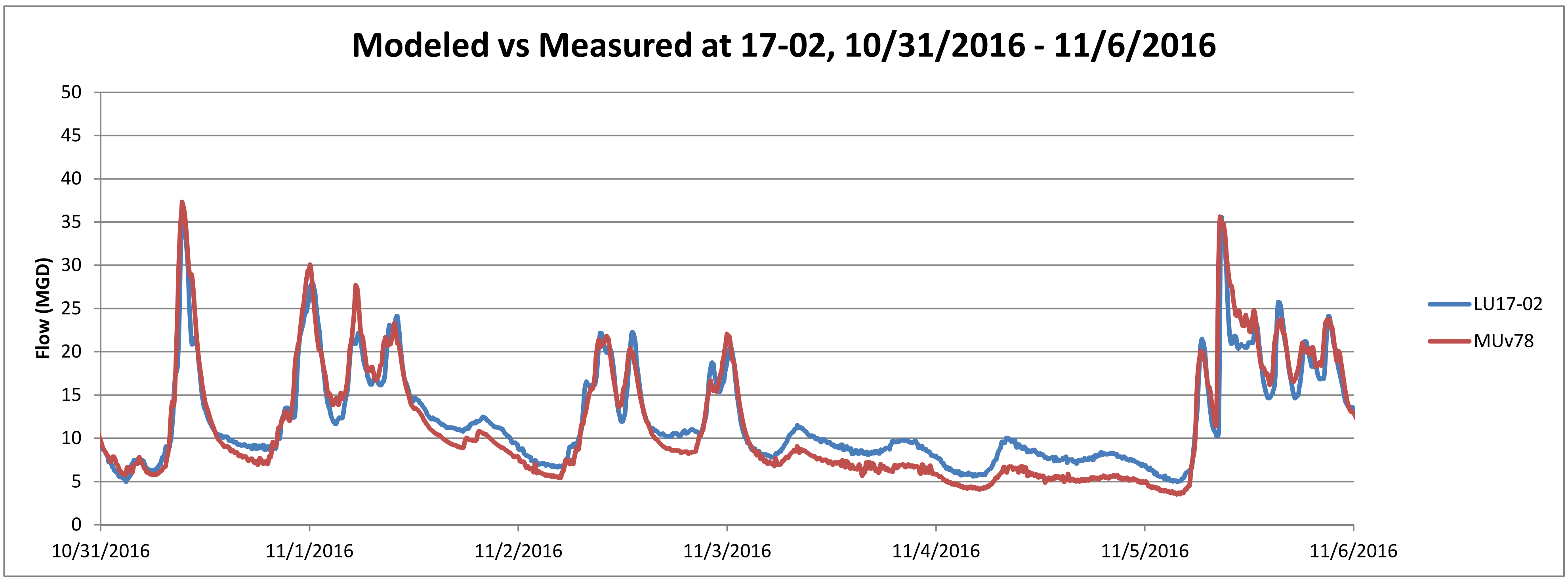
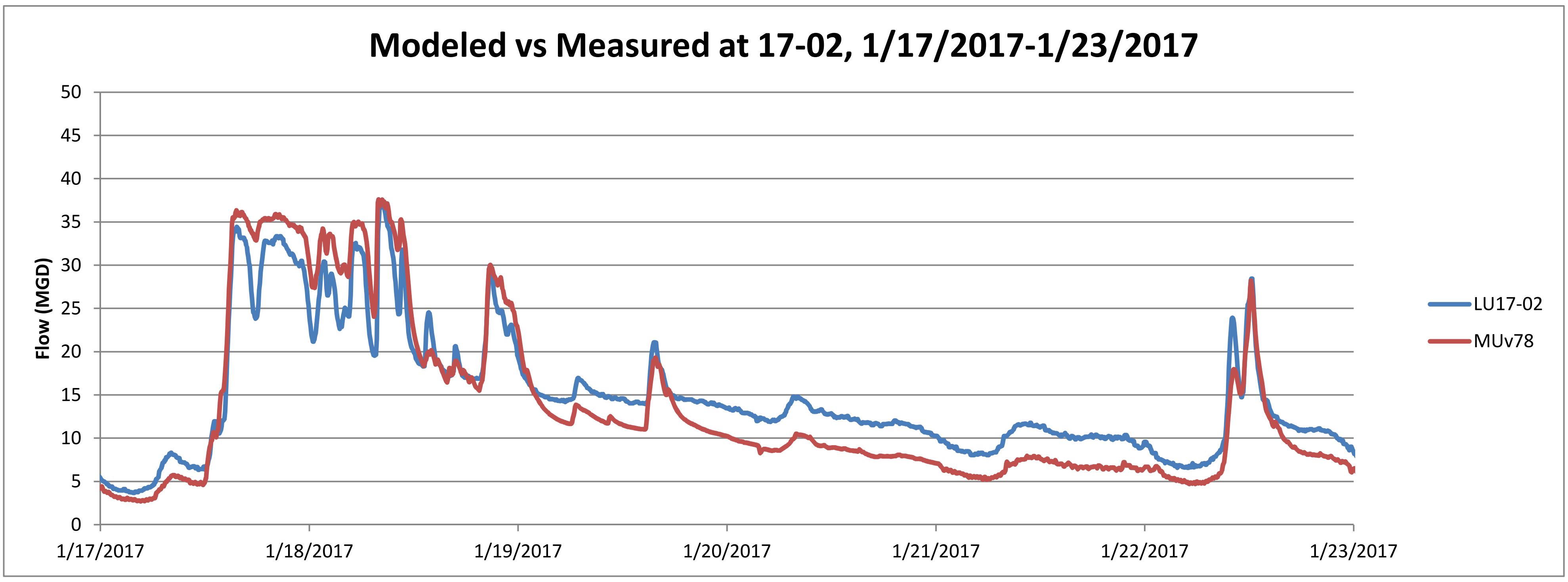
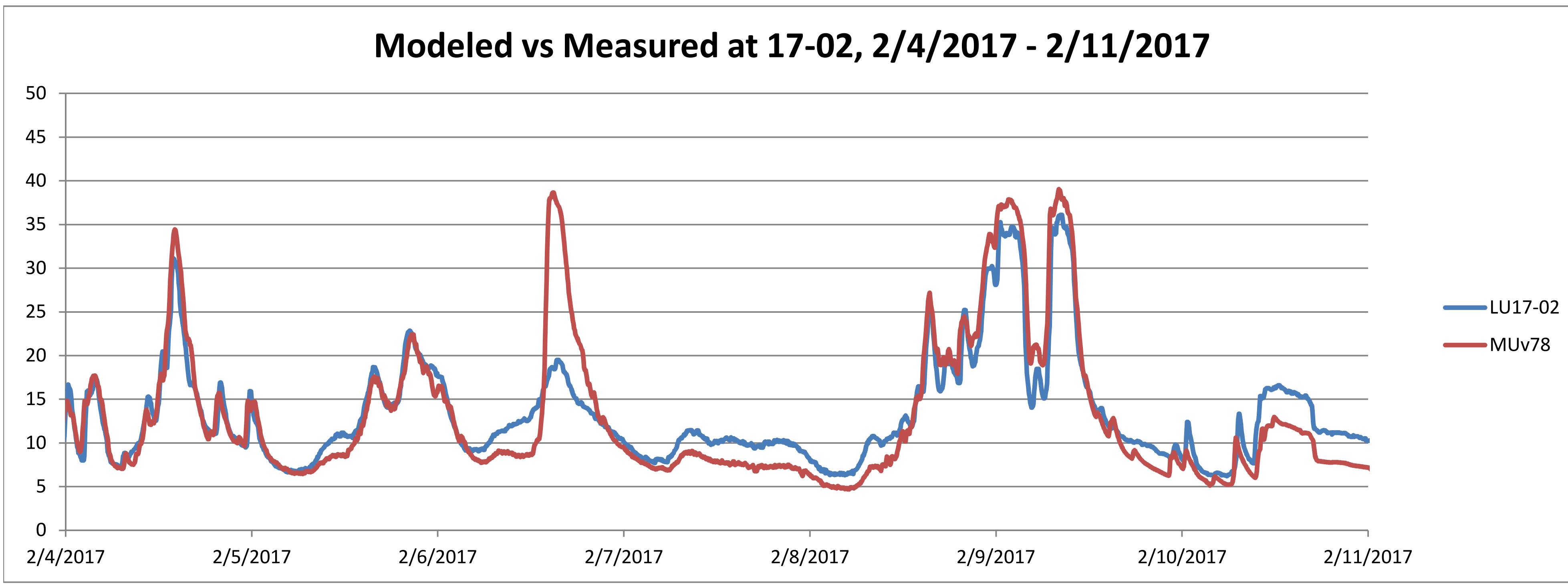
4/14/2017
4/15/2017
4/16/2017
4/17/2017
4/18/2017
11THAVE
MUv78

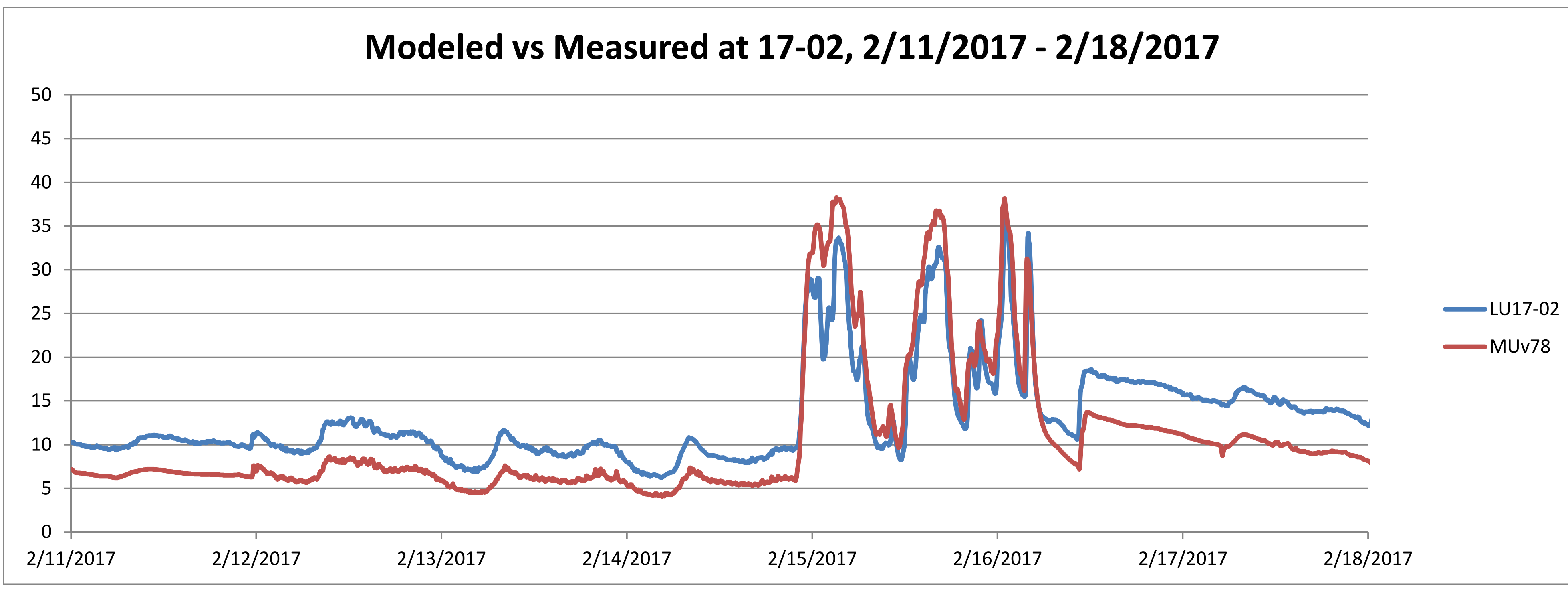

Modeled vs Measured at 17-02, 10/19/2016 - 10/25/2016
Flow (MGD)
LU17-02
MUv78
10/19/2016
10/20/2016
10/21/2016
10/22/2016
10/23/2016
10/24/2016
10/25/2016

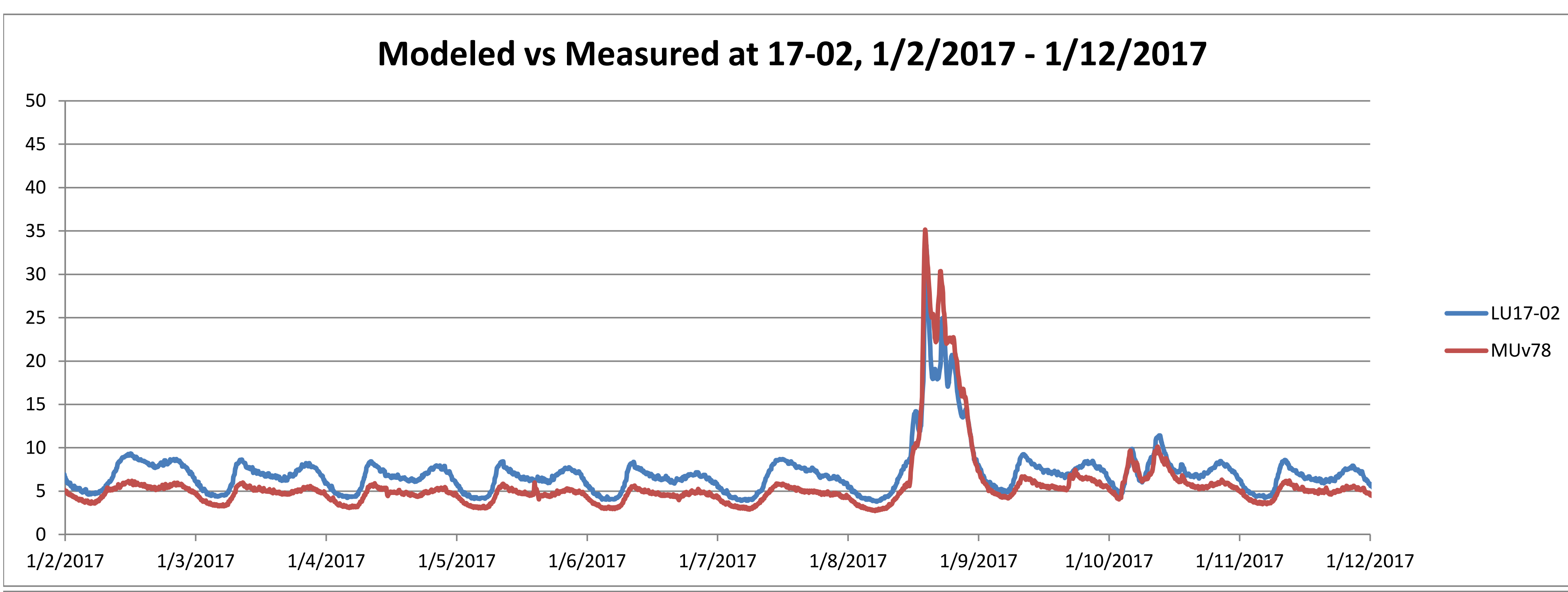

Modeled vs Measured at 17-02, 10/25/2016 - 10/31/2016
Flow (MGD)
LU17-02
MUv78
10/25/2016
10/26/2016
10/27/2016
10/28/2016
10/29/2016
10/30/2016
10/31/2016

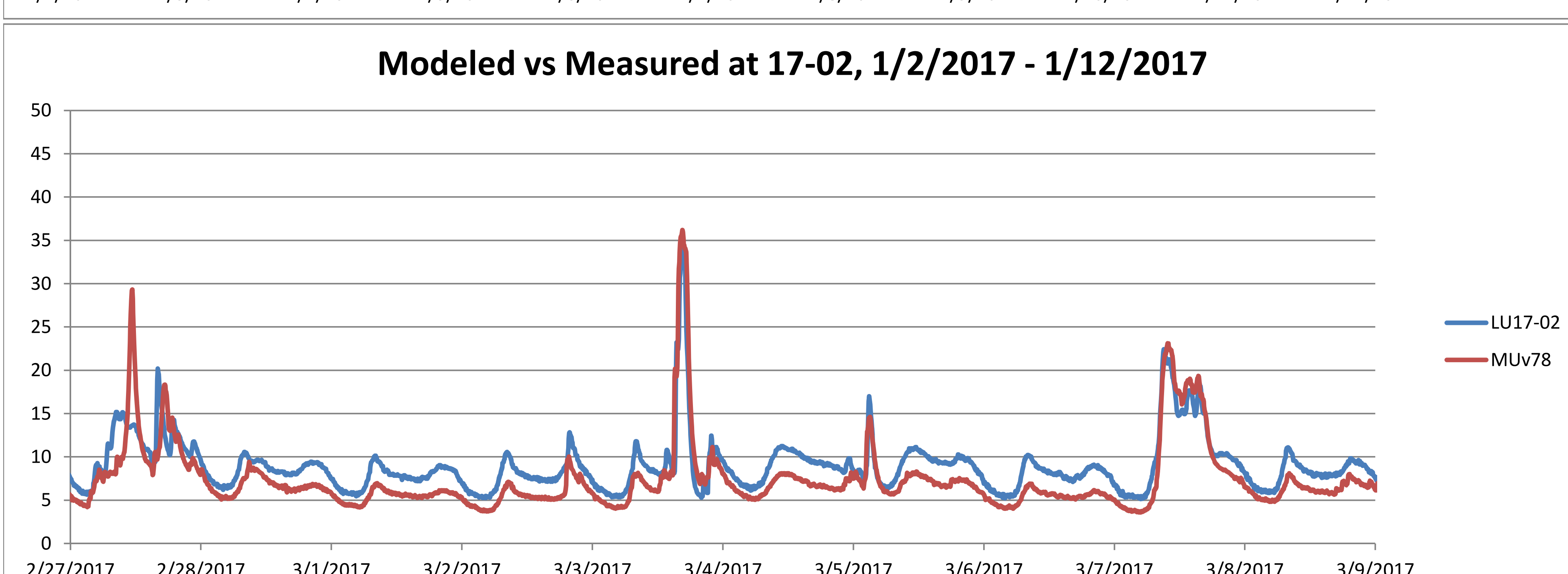

Modeled vs Measured at 17-02, 10/31/2016 - 11/6/2016
Flow (MGD)
LU17-02
MUv78
10/31/2016
11/1/2016
11/2/2016
11/3/2016
11/4/2016
11/5/2016
11/6/2016

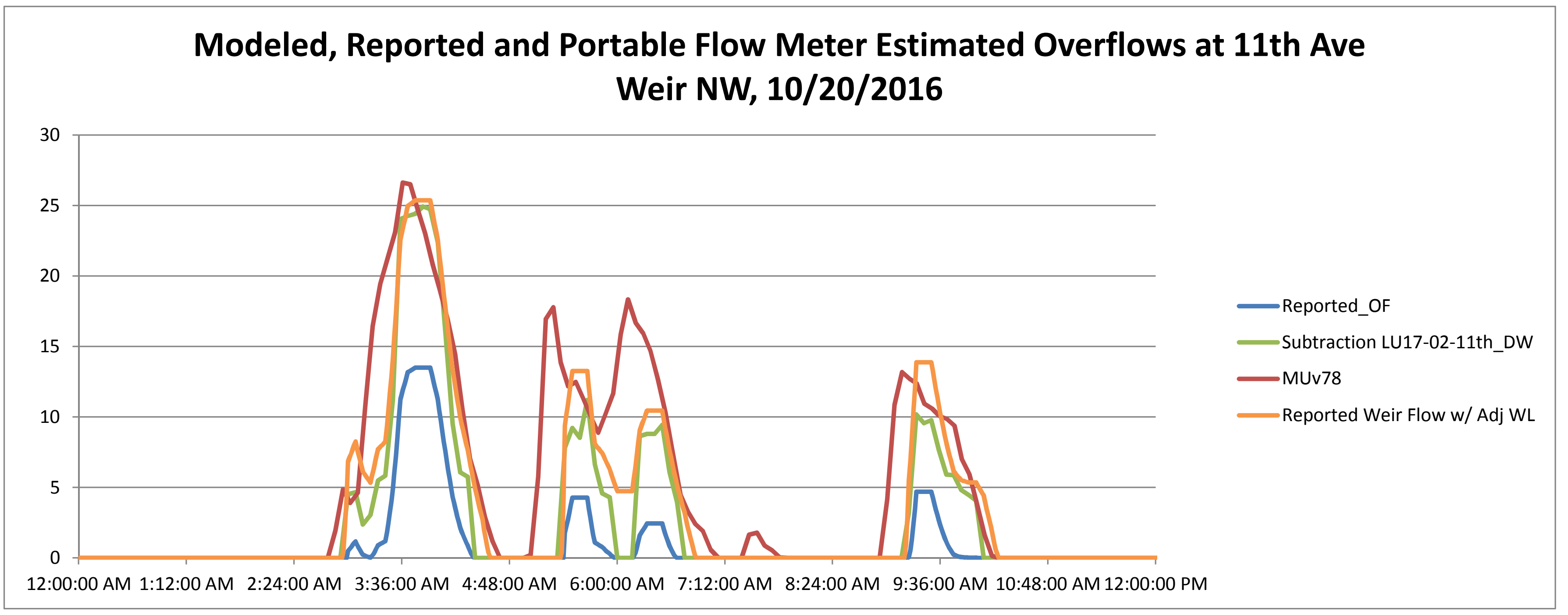

Modeled vs Measured at 17-02, 1/17/2017-1/23/2017
Flow (MGD)
LU17-02
MUv78
1/17/2017
1/18/2017
1/19/2017
1/20/2017
1/21/2017
1/22/2017
1/23/2017

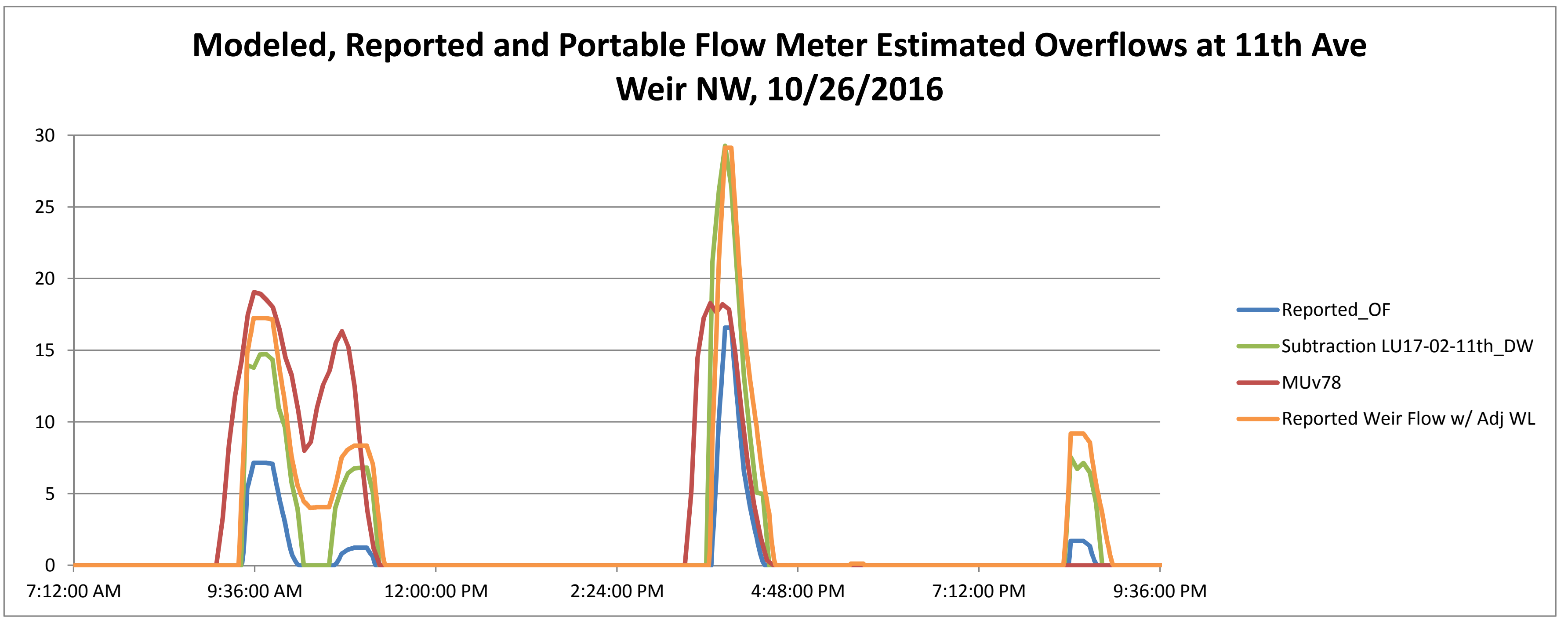

Modeled vs Measured at 17-02, 2/4/2017 - 2/11/2017
50
45
40
35
30
25
20
15
10
5
0
2/4/2017
2/5/2017
2/6/2017
2/7/2017
2/8/2017
2/9/2017
2/10/2017
2/11/2017
LU17-02
MUv78

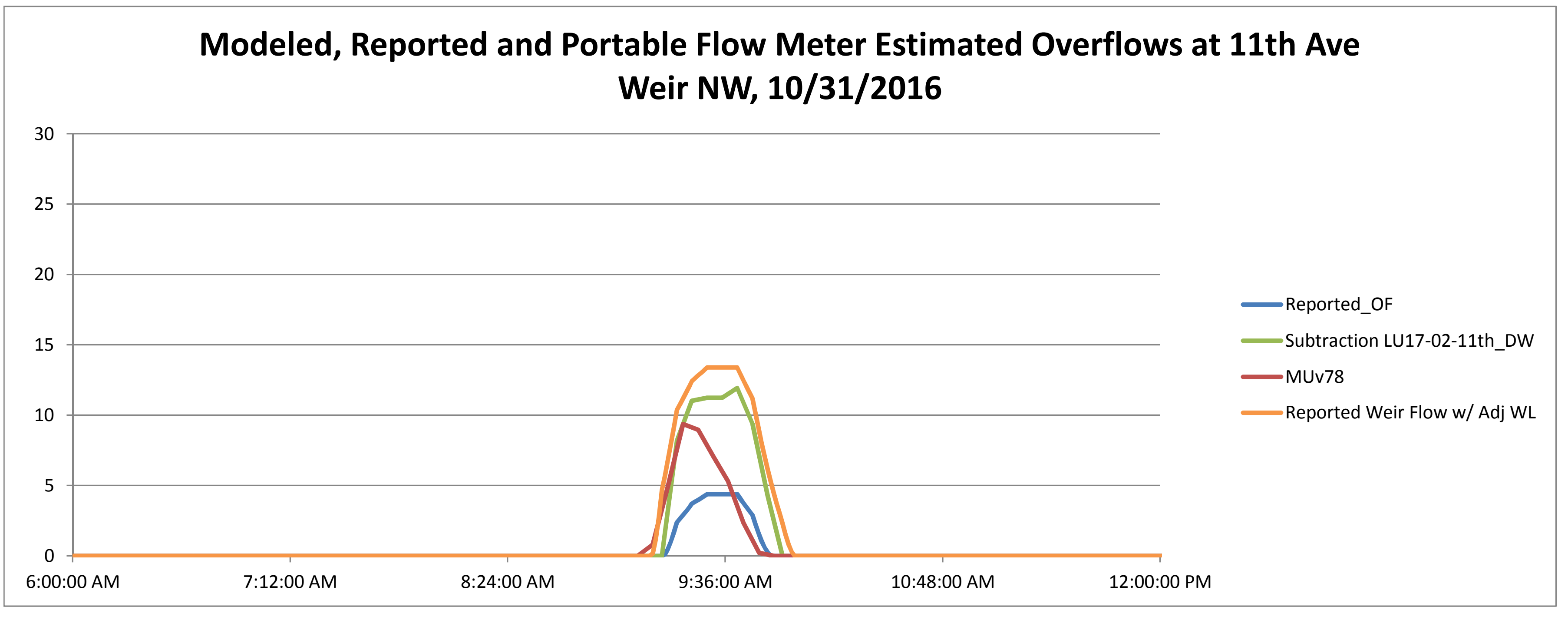

Modeled vs Measured at 17-02, 2/11/2017 - 2/18/2017
50
45
40
35
30
25
20
15
10
5
0
2/11/2017
2/12/2017
2/13/2017
2/14/2017
2/15/2017
2/16/2017
2/17/2017
2/18/2017
LU17-02
MUv78

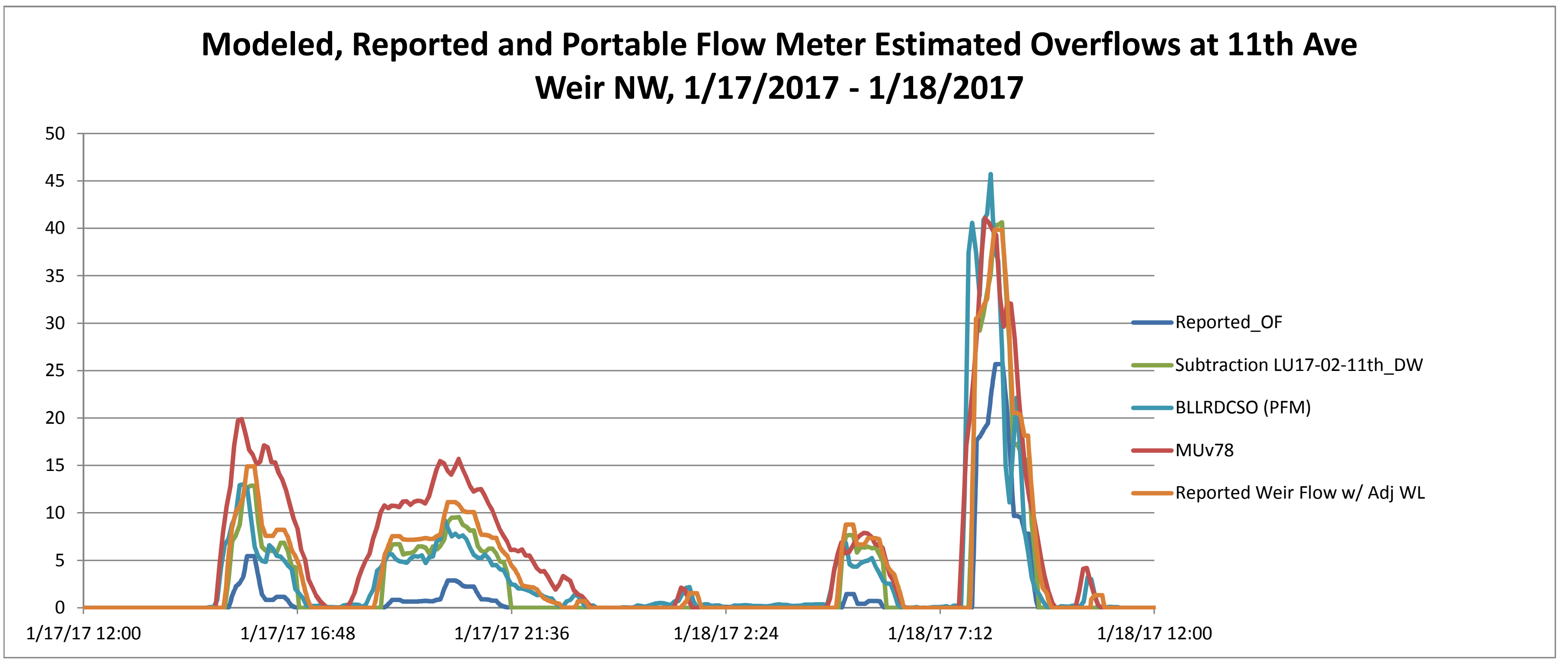

Modeled vs Measured at 17-02, 1/2/2017 - 1/12/2017
50
45
40
35
30
25
20
15
10
5
0
1/2/2017
1/3/2017
1/4/2017
1/5/2017
1/6/2017
1/7/2017
1/8/2017
1/9/2017
1/10/2017
1/11/2017
1/12/2017
LU17-02
MUv78

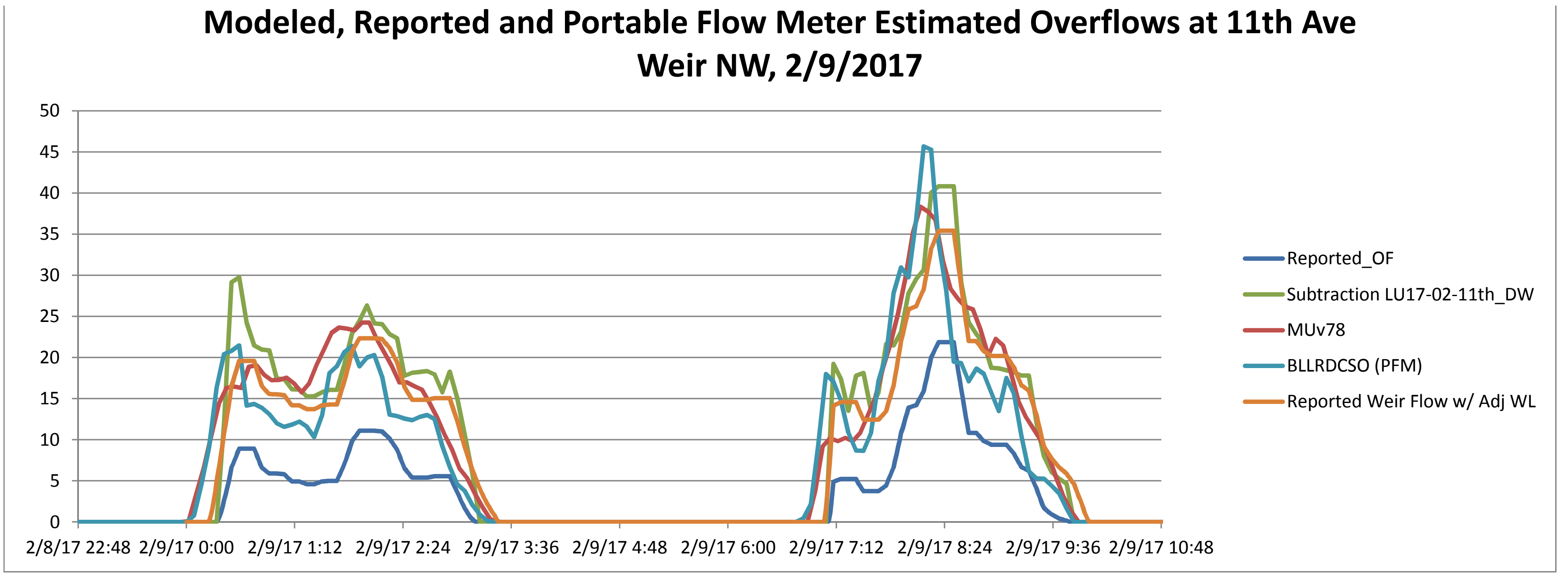

Modeled vs Measured at 17-02, 1/2/2017 - 1/12/2017
50
45
40
35
30
25
20
15
10
5
0
2/27/2017
2/28/2017
3/1/2017
3/2/2017
3/3/2017
3/4/2017
3/5/2017
3/6/2017
3/7/2017
3/8/2017
3/9/2017
LU17-02
MUv78


Modeled, Reported and Portable Flow Meter Estimated Overflows at 11th Ave Weir NW, 10/20/2016
30
25
20
15
10
5
0
12:00:00 AM
1:12:00 AM
2:24:00 AM
3:36:00 AM
4:48:00 AM
6:00:00 AM
7:12:00 AM
8:24:00 AM
9:36:00 AM
10:48:00 AM
12:00:00 PM
Reported_OF
Subtraction LU17-02-11th_DW
MUv78
Reported Weir Flow w/ Adj WL

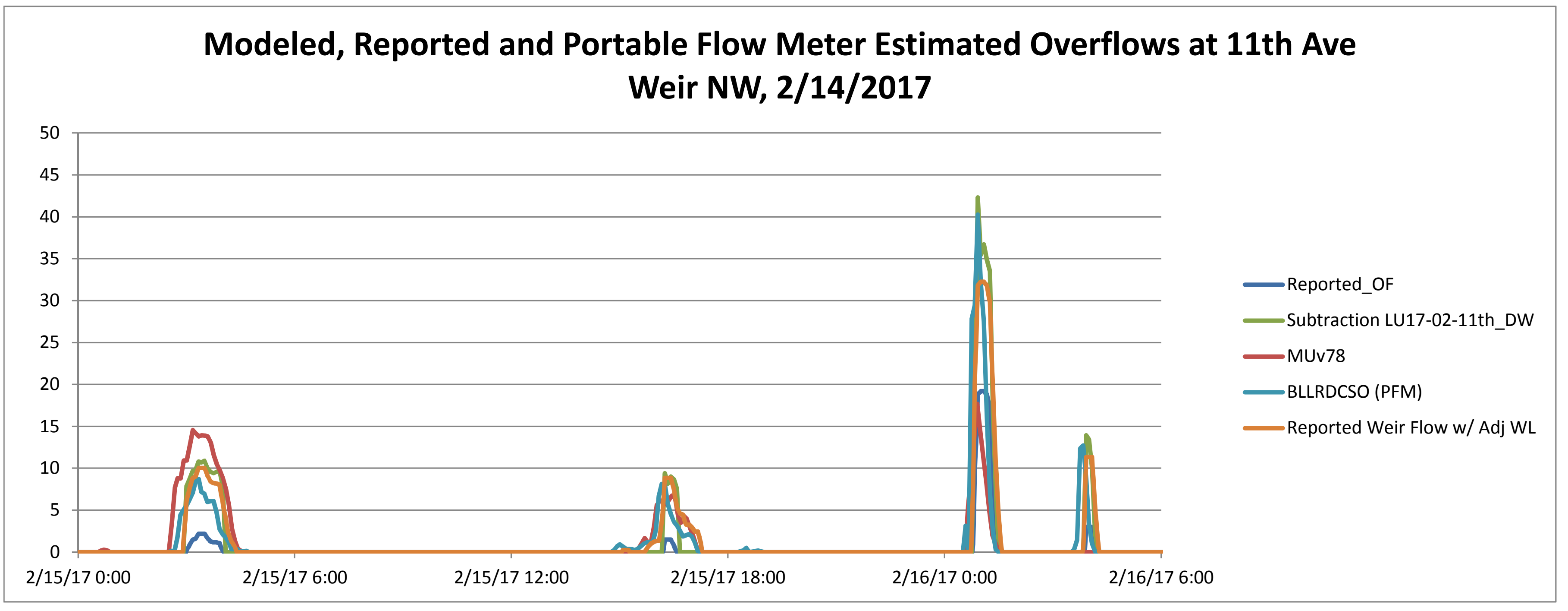

Modeled, Reported and Portable Flow Meter Estimated Overflows at 11th Ave Weir NW, 10/26/2016
30
25
20
15
10
5
0
7:12:00 AM
9:36:00 AM
12:00:00 PM
2:24:00 PM
4:48:00 PM
7:12:00 PM
9:36:00 PM
Reported_OF
Subtraction LU17-02-11th_DW
MUv78
Reported Weir Flow w/ Adj WL


Modeled, Reported and Portable Flow Meter Estimated Overflows at 11th Ave Weir NW, 10/31/2016
30
25
20
15
10
5
0
6:00:00 AM
7:12:00 AM
8:24:00 AM
9:36:00 AM
10:48:00 AM
12:00:00 PM
Reported_OF
Subtraction LU17-02-11th_DW
MUv78
Reported Weir Flow w/ Adj WL

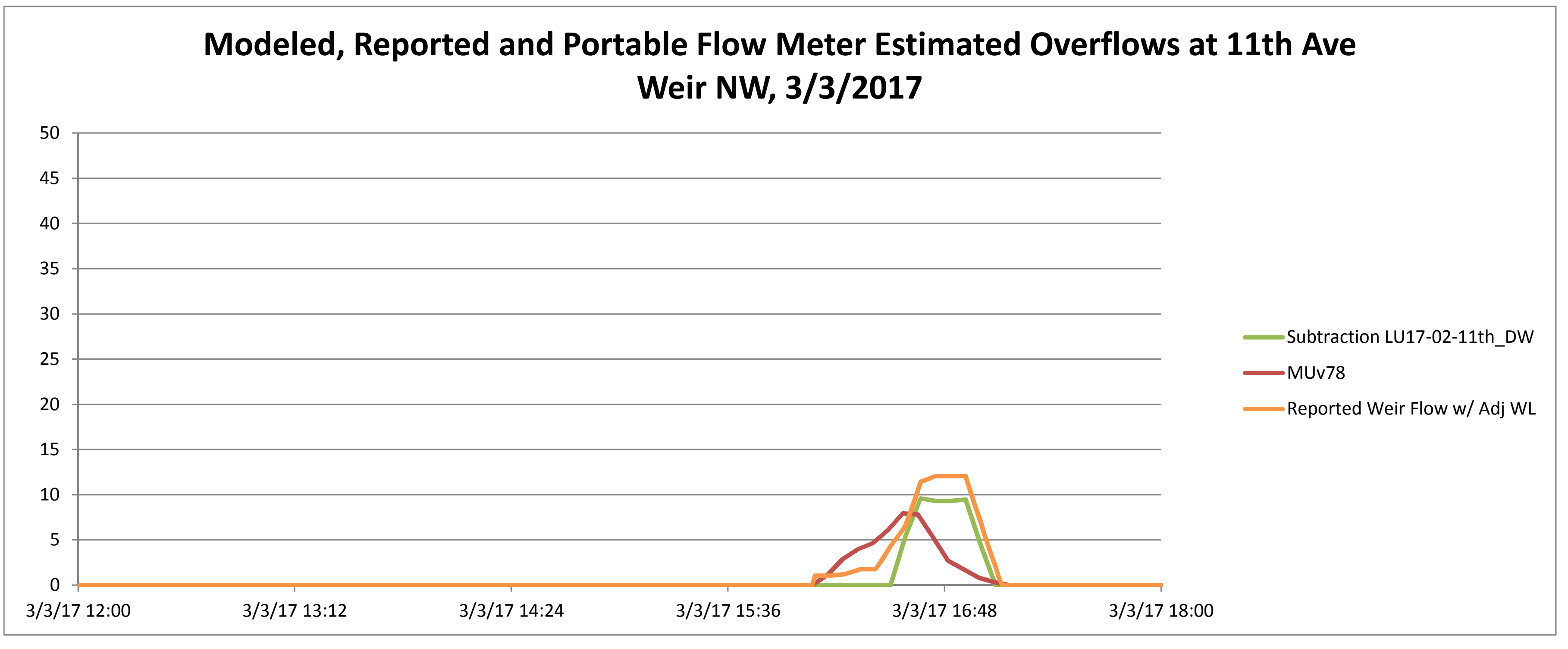

Modeled, Reported and Portable Flow Meter Estimated Overflows at 11th Ave Weir NW, 1/17/2017 - 1/18/2017
50
45
40
35
30
25
20
15
10
5
0
1/17/17 12:00
1/17/17 16:48
1/17/17 21:36
1/18/17 2:24
1/18/17 7:12
1/18/17 12:00
Reported_OF
Subtraction LU17-02-11th_DW
BLLRDCSO (PFM)
MUv78
Reported Weir Flow w/ Adj WL



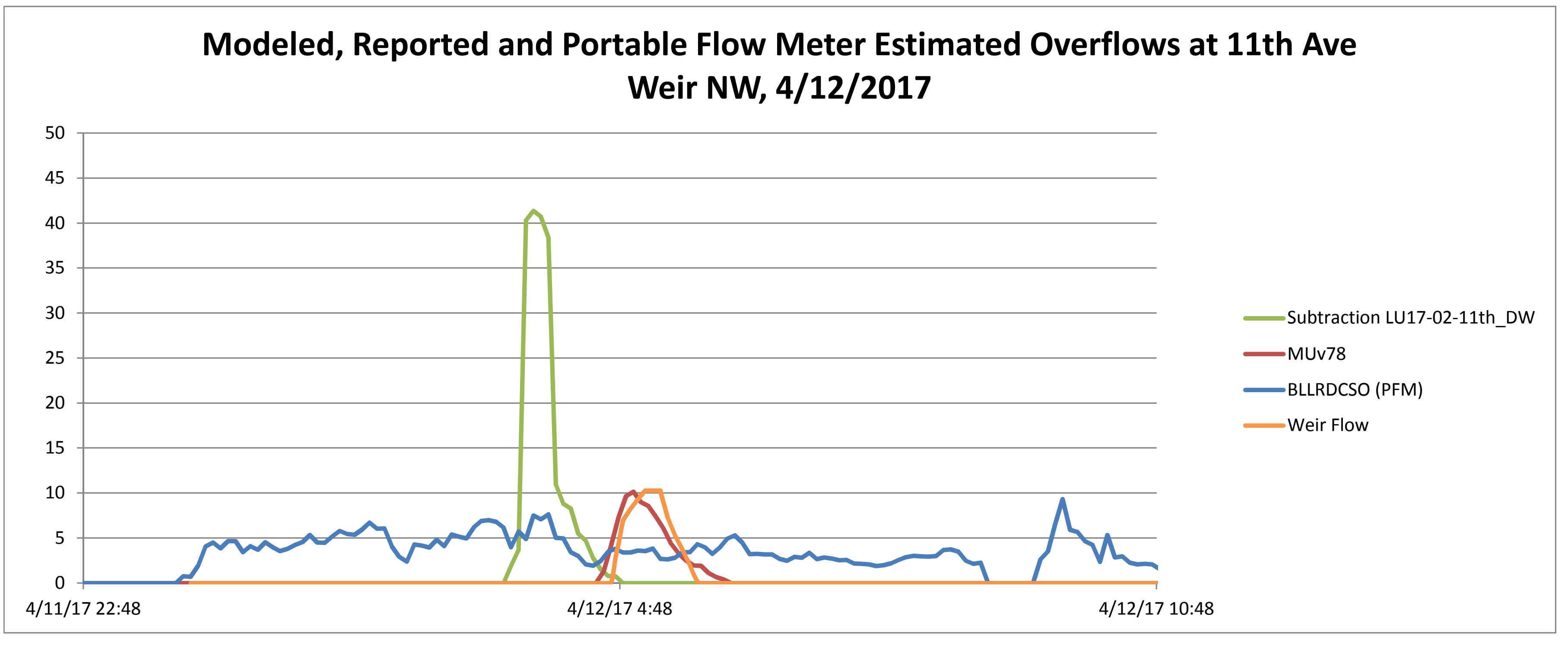
Modeled, Reported and Portable Flow Meter Estimated Overflows at 11th Ave Weir NW, 2/9/2017
50
45
40
35
30
25
20
15
10
5
0
2/8/17 22:48
2/9/17 0:00
2/9/17 1:12
2/9/17 2:24
2/9/17 3:36
2/9/17 4:48
2/9/17 6:00
2/9/17 7:12
2/9/17 8:24
2/9/17 9:36
2/9/17 10:48
Reported_OF
Subtraction LU17-02-11th_DW
MUv78
BLLRDCSO (PFM)
Reported Weir Flow w/ Adj WL



Modeled, Reported and Portable Flow Meter Estimated Overflows at 11th Ave Weir NW, 2/14/2017
50
45
40
35
30
25
20
15
10
5
0
2/15/17 0:00
2/15/17 6:00
2/15/17 12:00
2/15/17 18:00
2/16/17 0:00
2/16/17 6:00
Reported_OF
Subtraction LU17-02-11th_DW
MUv78
BLLRDCSO (PFM)
Reported Weir Flow w/ Adj WL



Modeled, Reported and Portable Flow Meter Estimated Overflows at 11th Ave Weir NW, 3/3/2017
50
45
40
35
30
25
20
15
10
5
0
3/3/17 12:00
3/3/17 13:12
3/3/17 14:24
3/3/17 15:36
3/3/17 16:48
3/3/17 18:00
Subtraction LU17-02-11th_DW
MUv78
Reported Weir Flow w/ Adj WL



Modeled, Reported and Portable Flow Meter Estimated Overflows at 11th Ave Weir NW, 4/12/2017
50
45
40
35
30
25
20
15
10
5
0
4/11/17 22:48
4/12/17 4:48
4/12/17 10:48
Subtraction LU17-02-11th_DW
MUv78
BLLRDCSO (PFM)
Weir Flow

Appendix E. Technical Memo: MIKE Urban North Interceptor Model Development and Calibration

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King County
Department of Natural Resources and Parks
Wastewater Treatment Division
King Street Center, KSC-NR-0500
201 South Jackson Street
Seattle, WA 98104-3855

MEMO

Date: January 31, 2018

TO: Ship Canal Water Quality Project Modeling Task Force
SPU – Eleanor Jackson, Ben Marre, Justin Twenter, Eset Alemu, Eric Habermeyer
KC WTD –John Komorita, Eric Mandel, Susan Kaufman-Una

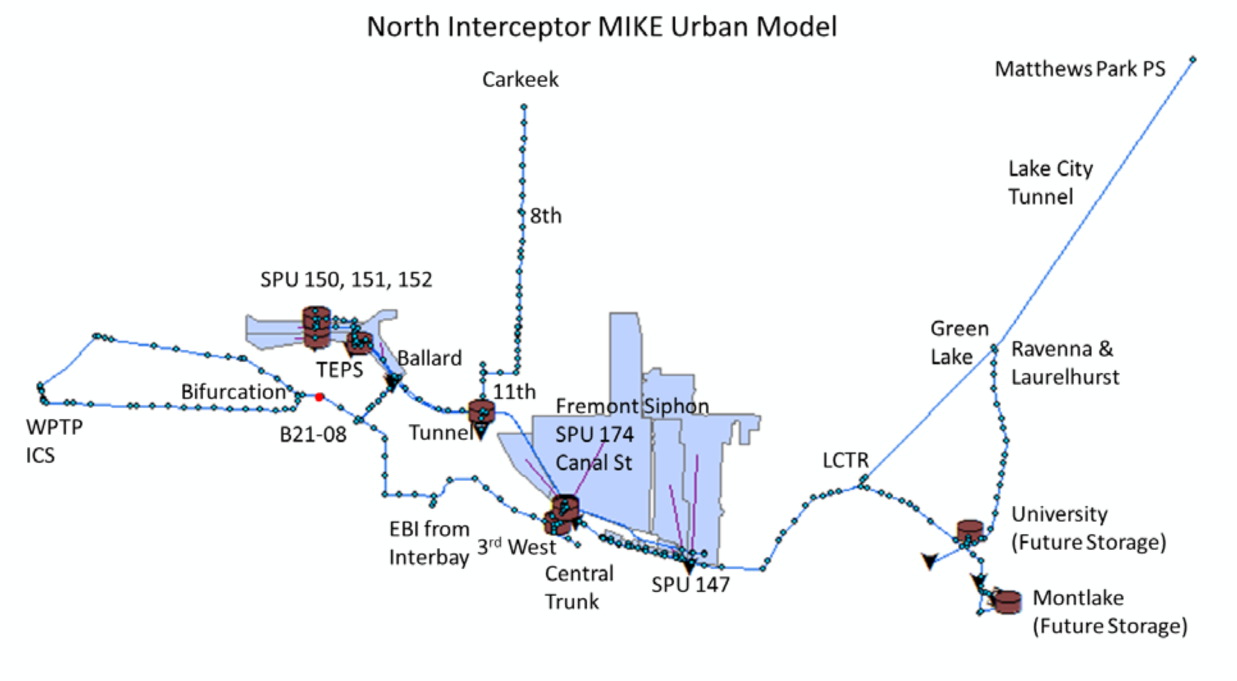
FM: Bruce Crawford, King County WTD Modeling Lead
Bob Swarner, King County WTD Modeling Supervisor

RE: **MIKE Urban North Interceptor Model Development and Calibration**

The North Interceptor hydraulic model was constructed through a joint effort of King County Wastewater Treatment Division (KC) and Seattle Public Utilities (SPU) modeling engineers using MIKE URBAN software. KC refined the model, adding the regulator gate controls at all King County facilities from University and Montlake Regulators at the east end to the West Point Treatment Plant at the West end. The overflow weirs at the SPU combined sewer overflow (CSO) structures at Ballard (Basins 150/151 and 152), at Wallingford and Fremont (Basins 147 and 174), and the KC weirs at 11th Ave. NW, 3rd Ave. w. and at Canal Street were added to the model. The purpose of the model was to simulate existing conditions and to simulate future conditions with the Ship Canal Water Quality Tunnel and planned upstream CSO storage projects integrated into the model. All the models were reviewed by SPU and KC staff and a consultant according to a standard QA/QC methodology to ensure proper construction and control algorithms. Figure 1 shows the extent of the North Interceptor model.

The Base Hydraulic Model

Development of the base North Interceptor hydraulic model was a joint effort between SPU and KC modelers, with each agency also providing review of the resulting model. The individual CSO structures at each CSO location were added to the model once the base North Interceptor was built. King County staff took that version of the model, and added the Real Time Control (RTC) statements necessary to emulate existing controls, and added simplified approximate versions of probable future King County storage projects. A plan view depiction of the final North Interceptor model is shown in Figure 2.



Note: ICS = Influent Control Structure; LCTR = Lake City Tunnel Regulator; TEPS = Tunnel Effluent Pump Station; B21-08 = North Interceptor level sensor location just downstream from Ballard Siphon

Figure 1 – Plan View Depiction of MIKE Urban North Interceptor Model

Inflow Sources

The model inflows were provided from a variety of basin models and SCADA sources. There has been an extensive collaborative effort over the past few years between KC and SPU to build and calibrate models

to simulate the hydrologic response from most of the area that drains to the North Interceptor. The model development and calibration has been documented in the following reports: Ballard Basins 150/151 and 152; 11th Ave. NW; Basin 174 (Fremont); and Basin 147 (Wallingford). Models have been previously calibrated by KC that includes most of the rest of the area draining to the North Interceptor.

Hydraulic Model Calibration Work

Once the hydraulic model was operating and inflows were provided, calibration of the hydraulic model to historic data commenced. The calibration effort, by KC staff, went through several iterations. Some of that work involved using additional code to simulate more complex existing controls (a dynamic-link library (DLL) written by Bruce Crawford of WTD used for control of Lake City Regulator). The calibration process used the Variable Manning's version 12 of the North Interceptor MIKE Urban model, which contains the old Ballard and Fremont Siphon characteristics, which were being used during the calibration period.

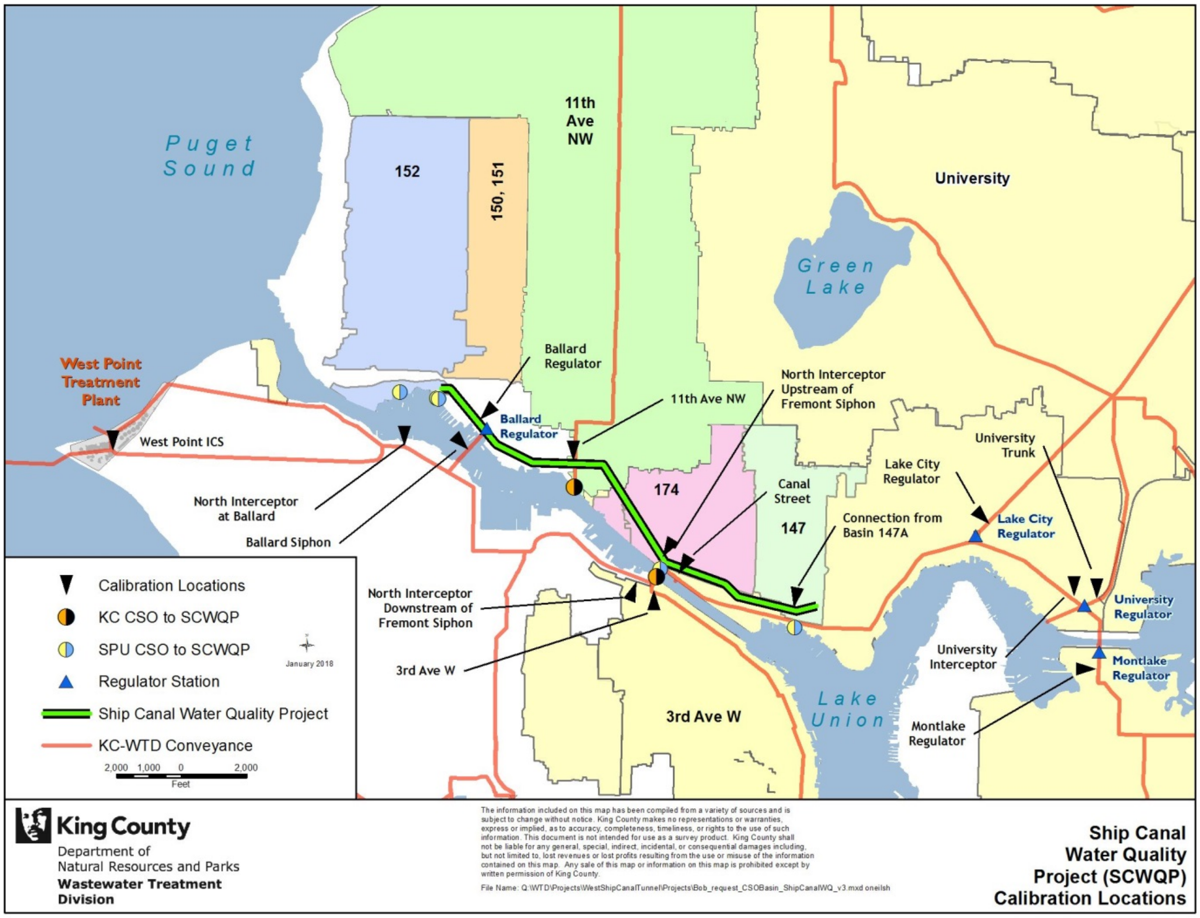


Figure 2 – Locations Where the North Interceptor Model was Calibrated to Level, Flow, and/or Overflow Data

The objective of the effort was to calibrate the model to match levels and flows for a time period with a high number of operating sensors, including SCADA and portable flow and level sensors, from both WTD and SPU data collection efforts. The period October 9, 2009 to February 1, 2010 was selected as the calibration period, since it included the time when there was data available from the greatest number of portable and permanent meters, and also contained several storm events.

The model has the old Ballard and Fremont siphons that were in place during the calibration period. The model uses SCADA data for the downstream West Point level as well as SCADA data for flow from Matthews Pump Station, Interbay Pump Station and Carkeek Pump Station. Use of the SCADA data, where available, reconciles flows to actual flows as much as possible for “out of area” inflows. It provides a closer look at “in area” flows and friction factors.

The Manning’s pipe friction was the main variable adjusted for reaches of pipes, though cross sections with sediment were added based on level information, CCTV videos of the pipes, and field personnel statements. Graphs for this calibration are provided below, under the heading “North Interceptor Model Calibration Results.”

Interaction and feedback from SPU modelers and Consultants was an ongoing and essential part of the calibration process.

Model Integration Work with the Ship Canal Water Quality Project (SCWQP) Tunnel Model

Once the calibration was sufficiently advanced, the newer Ballard and Fremont Siphon configurations were added to the model. Then the model was provided to the SPU and their consultants for integration with the SCWQP tunnel model, with connecting conveyance pipes. Once the addition of those elements was completed, the Integrated Tunnel Model was provided to WTD for review and further refinement of the controls. SPU staff and their consultants provided valuable review and feedback to ensure the model and associated control algorithms achieved the desired results. The Integrated Tunnel Model, with the real-time control (RTC) components (with a DLL only for Lake City Regulator), was provided to SPU and their consultants for their use in conducting long term simulations.

North Interceptor Model Calibration Results

The hydraulic model calibration process consisted of calibrating pipe friction, sediment and minor losses to match levels and flow distributions measured over a four month period, October 2009 through February 1, 2010. The period was selected was one having a large amount of measured data available during a period with a good distribution of storm events.

Locations used for model calibration are shown in Figure 2. Graphs comparing model data to measured and calculated data for the four month calibration period are shown below. The results are from the North Interceptor model variable Manning's version 12. The graphs show the final hydraulic calibration results. These graphs were used, along with an understanding of the limitations of measured data, as well as video inspection and field crew information, to perform adjustments of the model friction, sediment and minor losses.

There are three sets of graphs. Set one looks at levels from the downstream end to the upstream end of the model area. Set two looks at flow rates from the downstream end to the upstream end of the model area. The third and final set of graphs present the comparison of model results with estimated overflow events.

Level Calibration Graphs

Figures 3 through 14 show the comparison of simulated water surface levels at various locations in the calibrated North Interceptor Model with the measured levels from permanent or portable level sensors in the North Interceptor system.

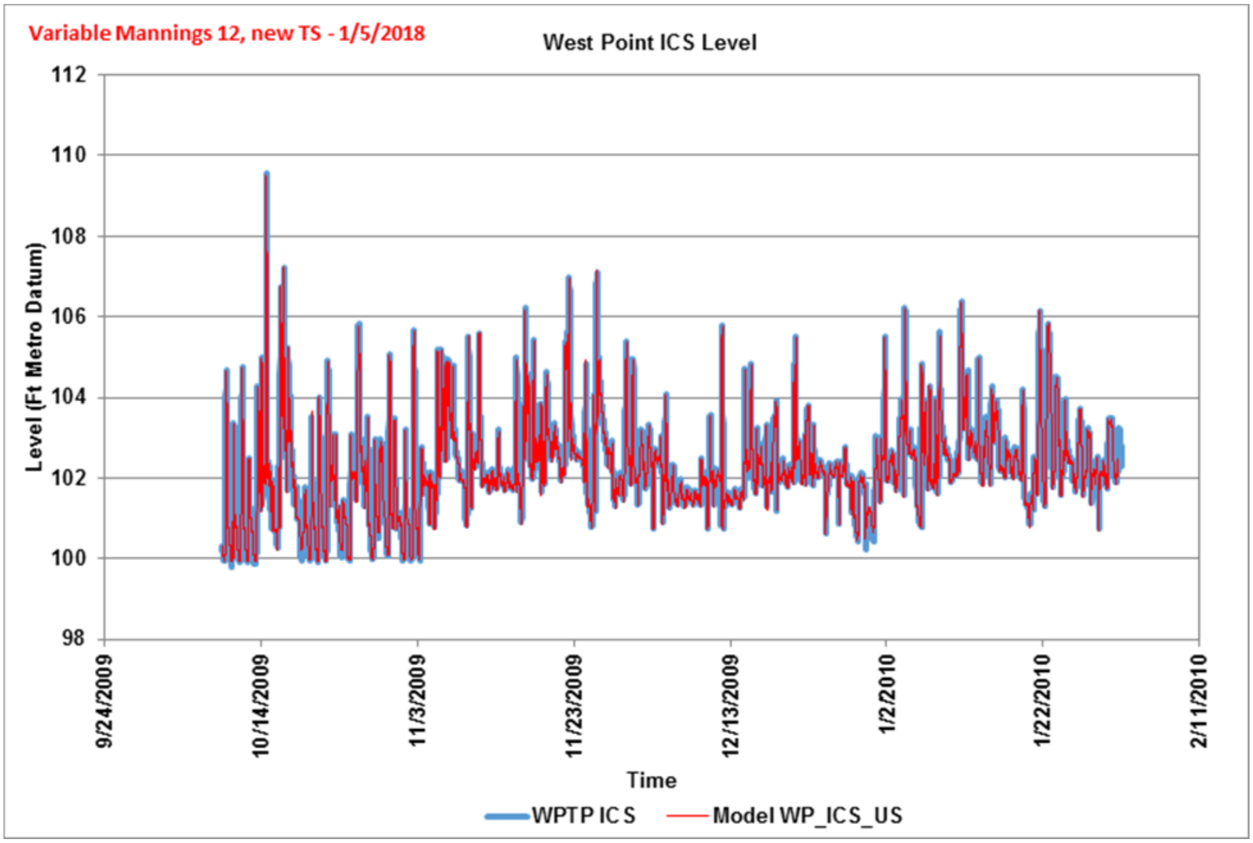


Figure 3 – Water Surface Elevation in the West Pont Influent Control Structure

The SCADA level in the West Point Influent Control Structure (ICS) was used as the model downstream boundary condition, so the model tracks the SCADA exactly.

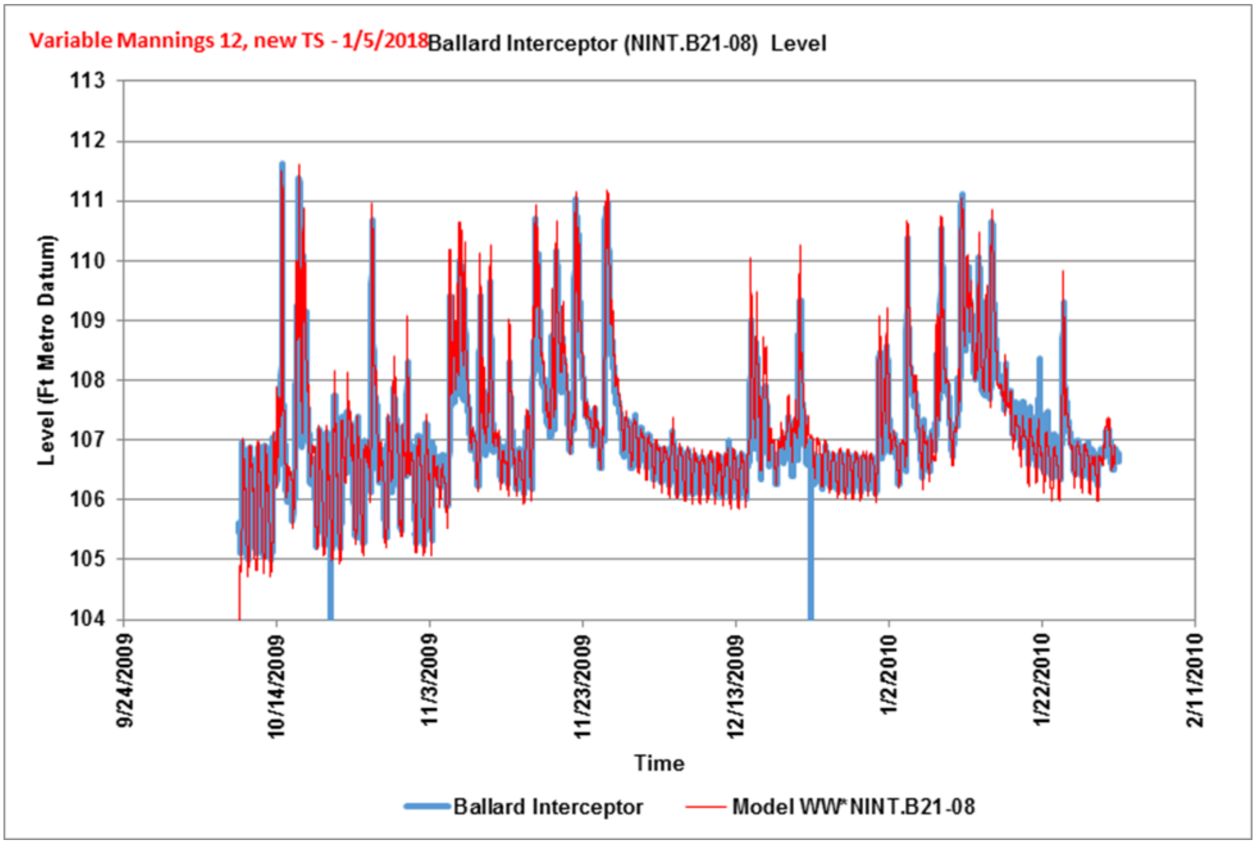


Figure 4 – Water Surface Elevation in the North Interceptor at Ballard

This graph shows the measured and modeled water surface elevations in the North Interceptor at the location from which the Ballard Regulator Gate is controlled. The friction adjustments from West Point through the Magnolia tunnels are matching most storm events. Note that some deviations occur when the flow provided from the hydrologic models does not match the actual system flows. (See the flow graphs for that information.) Adjustment of model settings typically requires consideration from many graphs for each adjustment proposed.

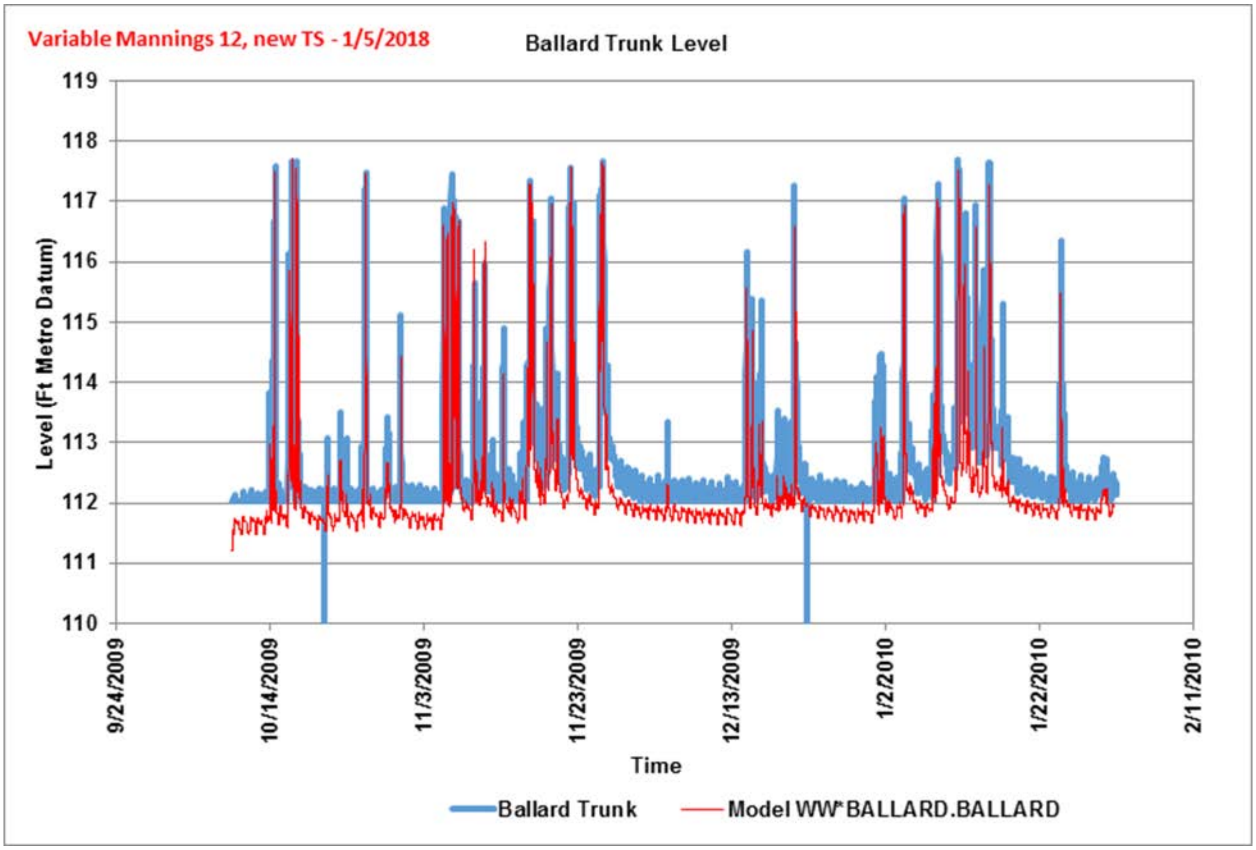


Figure 5 – Water Surface Elevation in the Ballard Trunk

The measured data for this graph is from the SCADA sensor which cannot measure below a depth of 6 inches in the channel. As with any sensor, there can be issues with the physical sensor placement and with the electronic sensor adjustments. The calibration process focused on matching the peak levels as closely as possible, since the upstream modeled areas had been calibrated to this sensor data and required matching performance to achieve consistent results.

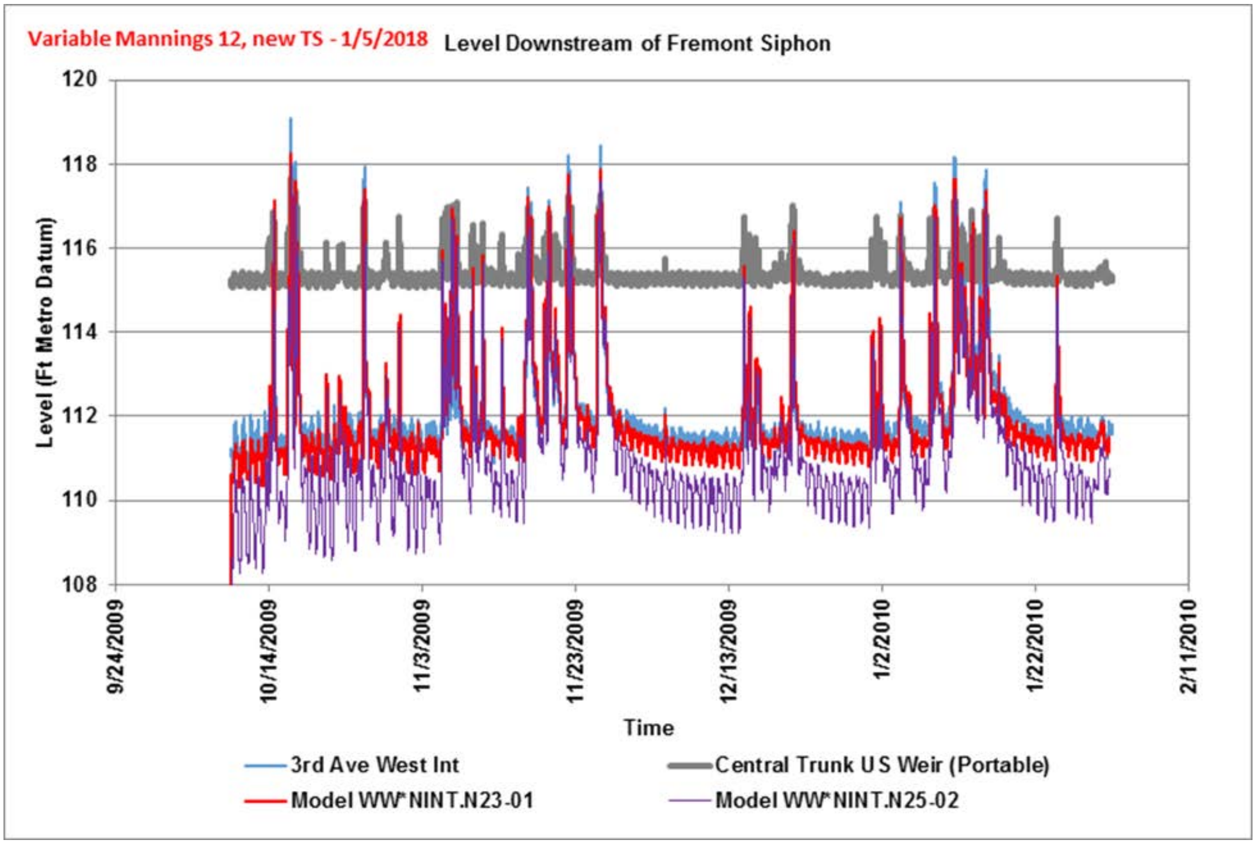


Figure 6 – Water Surface Elevation in the North Interceptor Just Downstream of the Fremont Siphon

The 3rd Ave West "Interceptor" sensor was found to be providing level measurements above those of upstream sensors, including a SPU portable meter placed upstream of 3rd Ave West during this time period (gray). The calibration process focused on tuning the model closer to that SPU meter data as it provided more consistent results for graphs below.

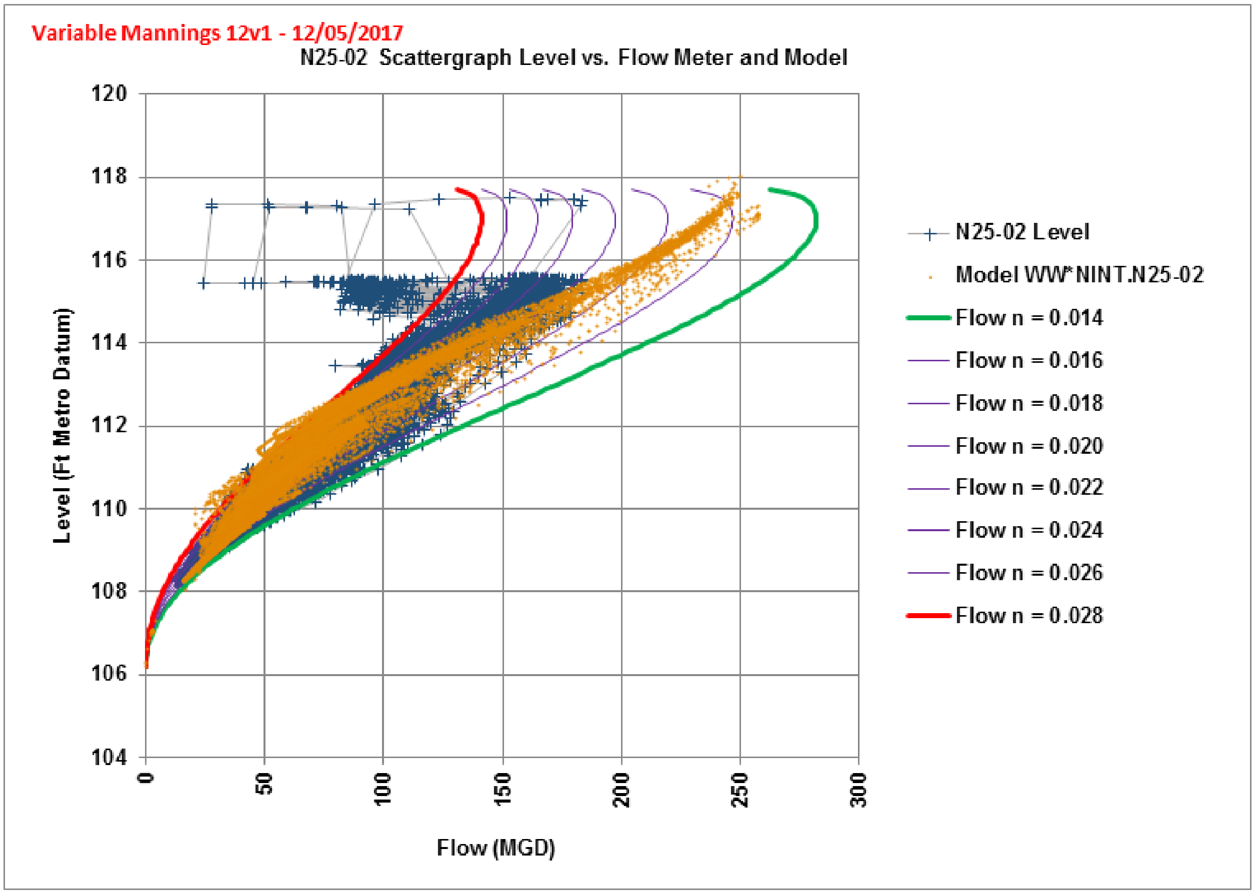


Figure 7 – Scattergraph of Level vs. Flow Downstream of Fremont Siphon and Third Ave West

Meter data at Maintenance Hole (MH) N25-02, which is one MH downstream of the confluence between the Central Trunk and the North Interceptor, was available from 8/30/2016 to 1/31/2017. This data was used to help tune the downstream friction properly in spite of the excessively high levels noted for a sensor in Figure 6. Scattergraph plots of the measured data (8/16 – 1/17) and the model output (10/09 – 2/10) were compared in Figure 7. The data points reflecting lower flow rates at higher water surface elevations are likely the result of the ICS level being different than the levels assumed in the model. Note that the blank band in the measured data between elevation 115.5 and 117.5 is a sensor issue.

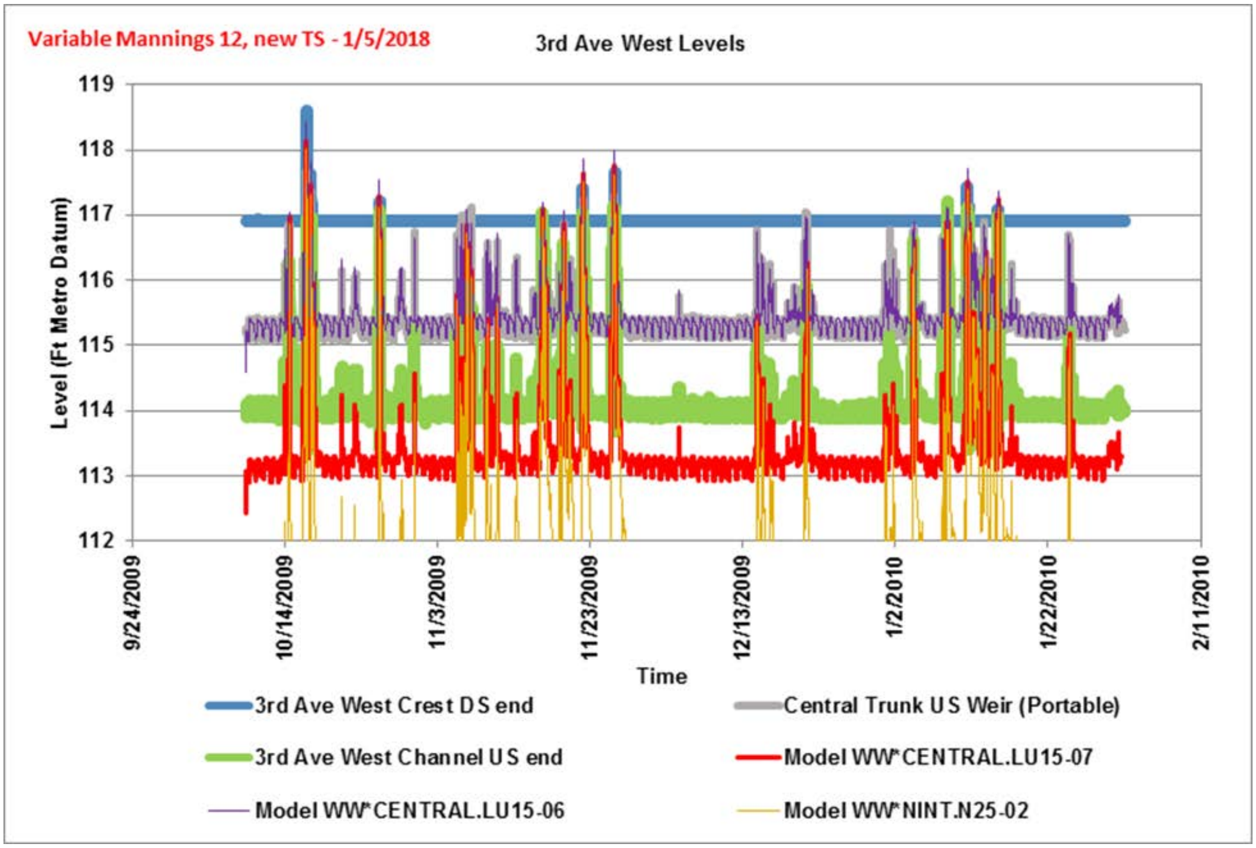
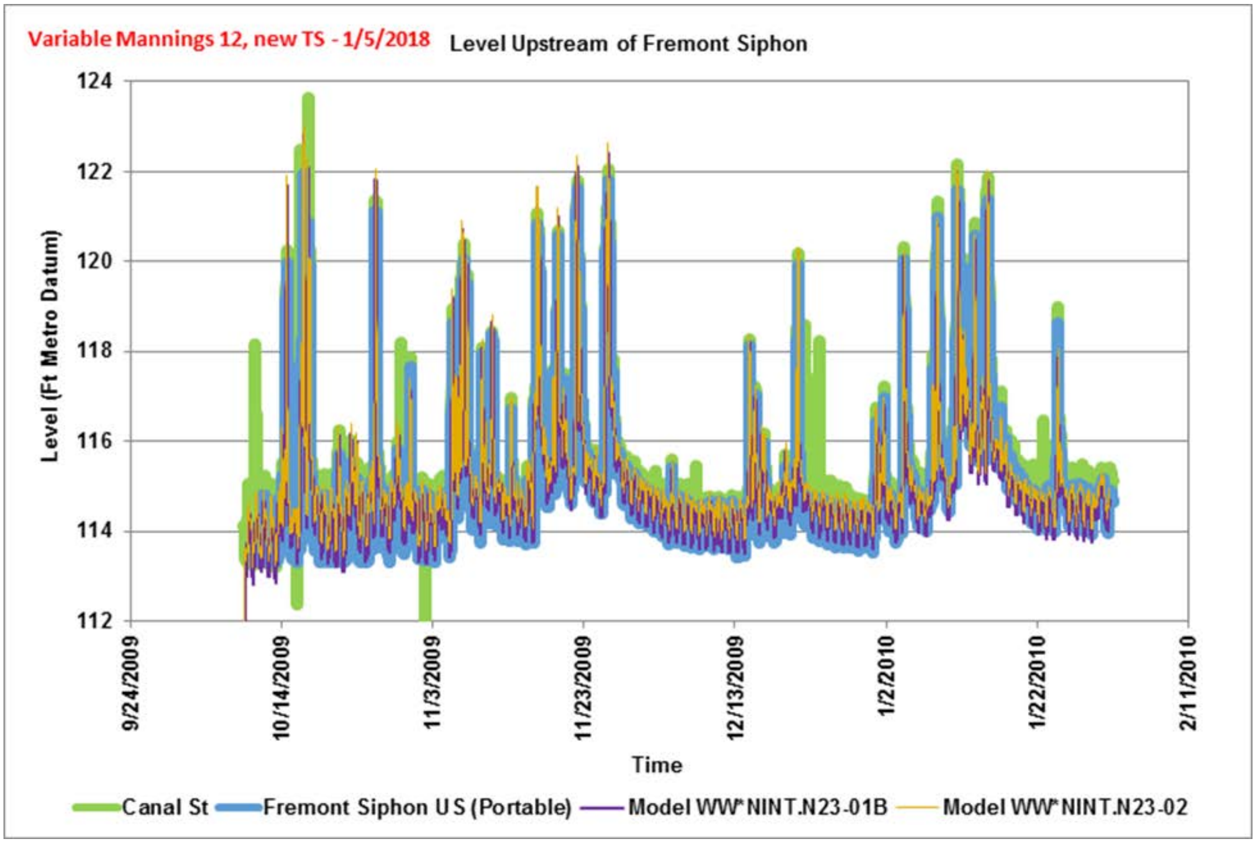


Figure 8 – Water Surface Elevation near Third Ave West

This graph contains data from three model locations and three level sensors in close proximity. Note that the modeling institutional knowledge includes the fact that this weir has been observed with a traveling hydraulic jump along its length. Mike Urban is not suited for detailed modeling of the traveling hydraulic jump. Indications from modeling are that the jump would travel upstream relatively quickly when compared to the model time step and storm duration.

Another issue at this location is partial occlusion of the outfall from rip rap placement in the Ship Canal which was observed on an outfall inspection video. Since the model does not include the occlusion, it may tend to have lower levels during some overflows (especially as the overflow increases). However, the event frequency matches for modeled and measured data.



Note: MH WW*NINT.N23-02 corresponds to the Canal St. weir location; WW*NINT.N23-01B corresponds to the Fremont Siphon U S (portable) location

Figure 9 – Water Surface Elevation Upstream of the Fremont Siphon

This location was used for tuning the Fremont Siphon friction losses. Note that the modeled levels tend to run a bit high at this location in order to obtain better calibration for overflows for Basin 147 and for KC's Canal St. weir.

The event with the highest measured level spike had flows in excess of that provided by the model, plus a SCADA issue that impacted the quality of the SCADA data for the event. As such, the hydraulics of the system was not tuned to that single event.

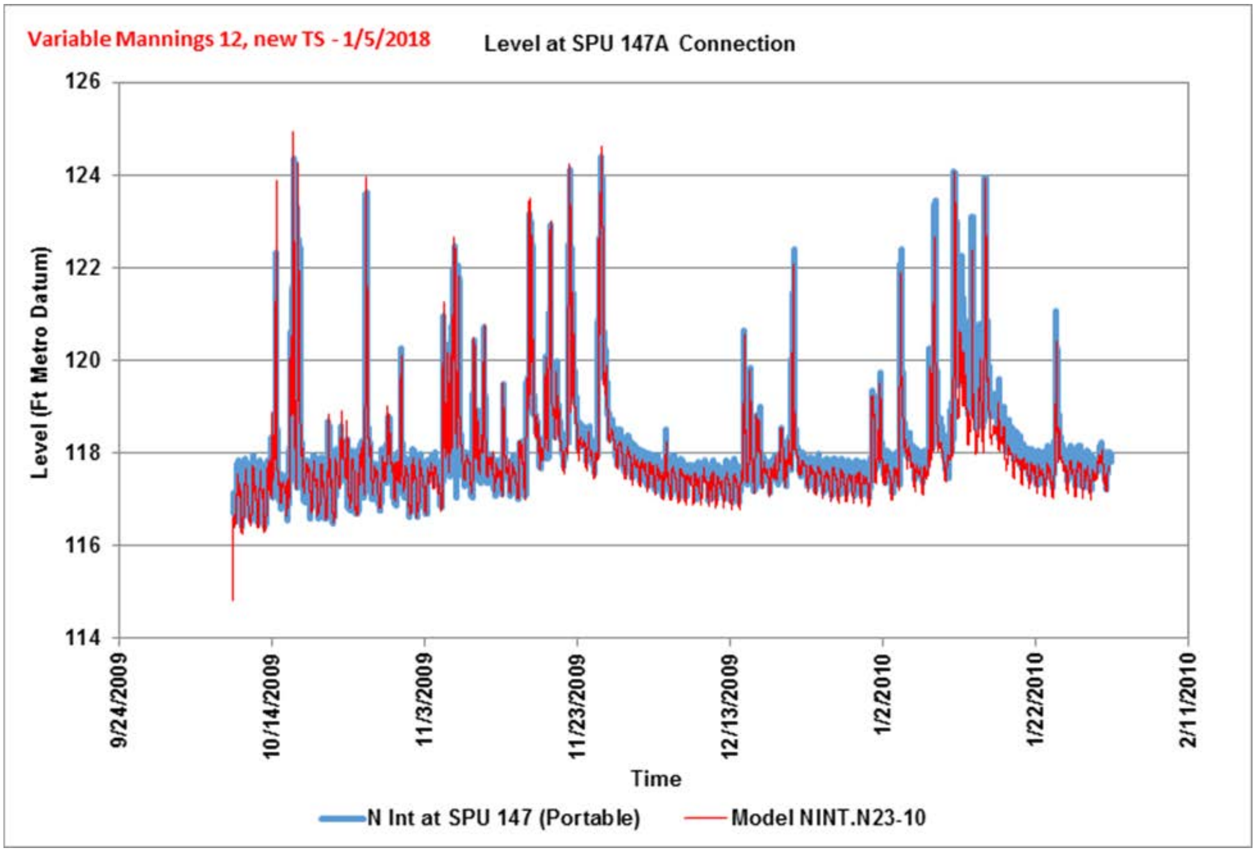


Figure 10 – Water Surface Elevation at Sub-basin 147A Connection

Figure 10 shows the level in the North Interceptor at the connection point for SPU Sub-basin 147A. The model is running somewhat high at peak of storm events in the early part of this simulation, which tends to help the overflow volumes match.

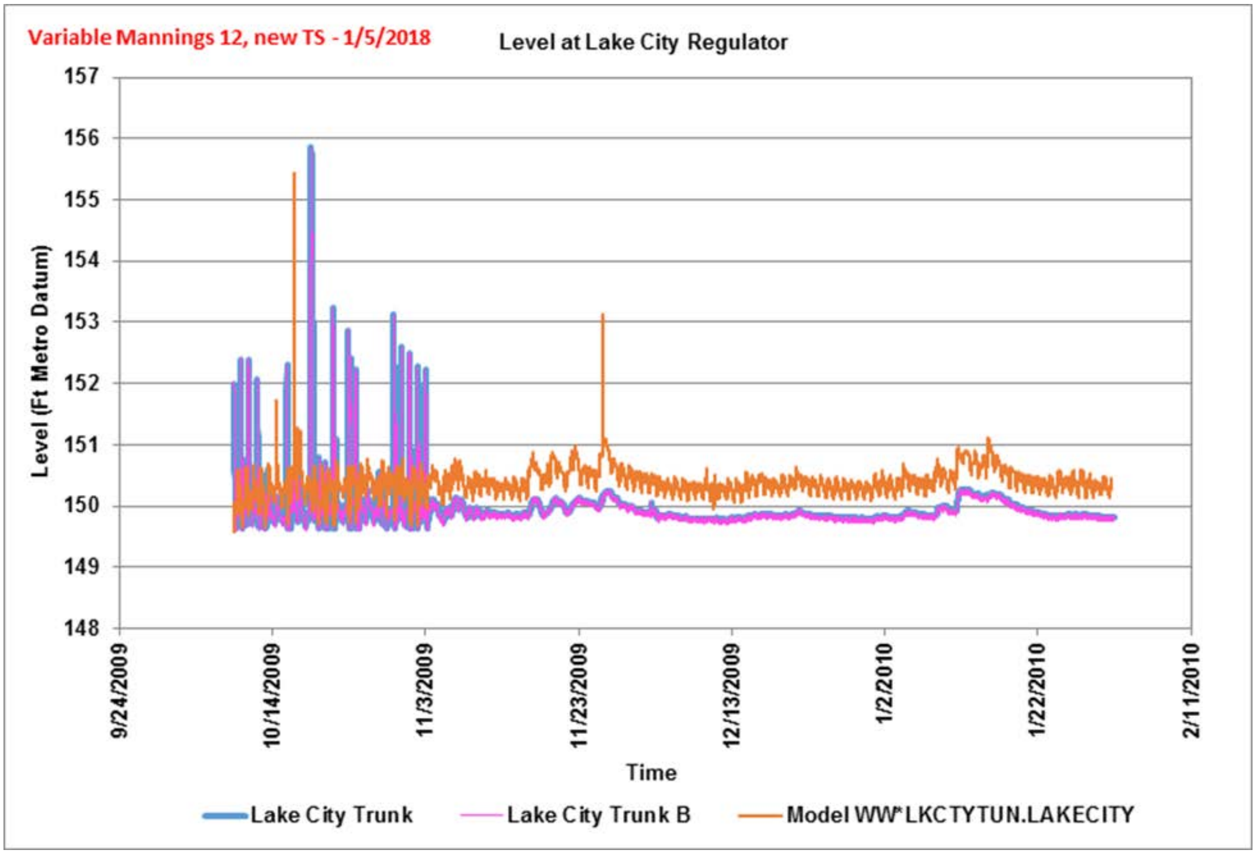


Figure 11 – Water Surface Elevation in the Lake City Tunnel Regulator Trunk

The model is not matching the SCADA level sensor data at Lake City Tunnel Regulator. However, KC staff believe that the level sensors are not reading the levels accurately. WTD crews are investigating the sensor installations to identify and resolve discrepancies.

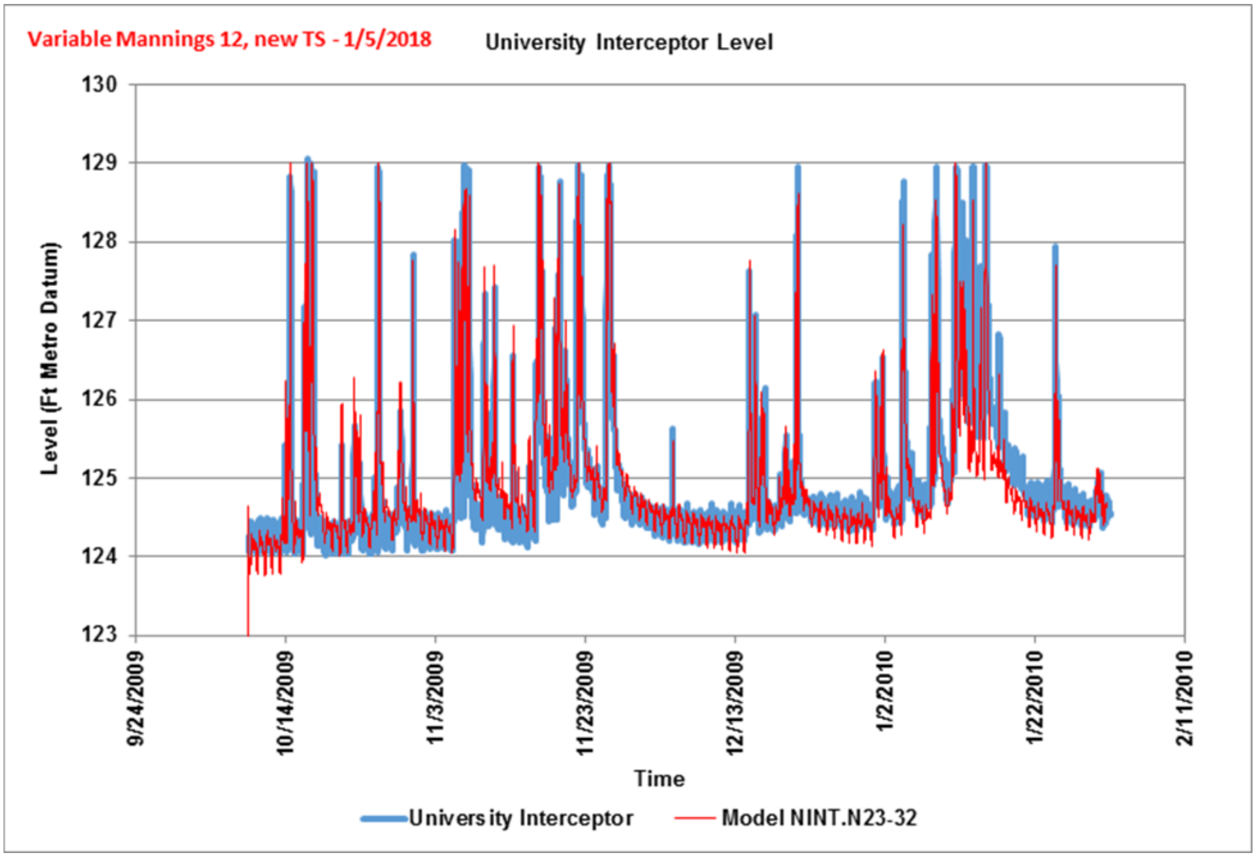


Figure 12 – Water Surface Elevation downstream of the University Regulator

This graph shows the level in the North Interceptor just downstream of the University Regulator (called the University Interceptor level).

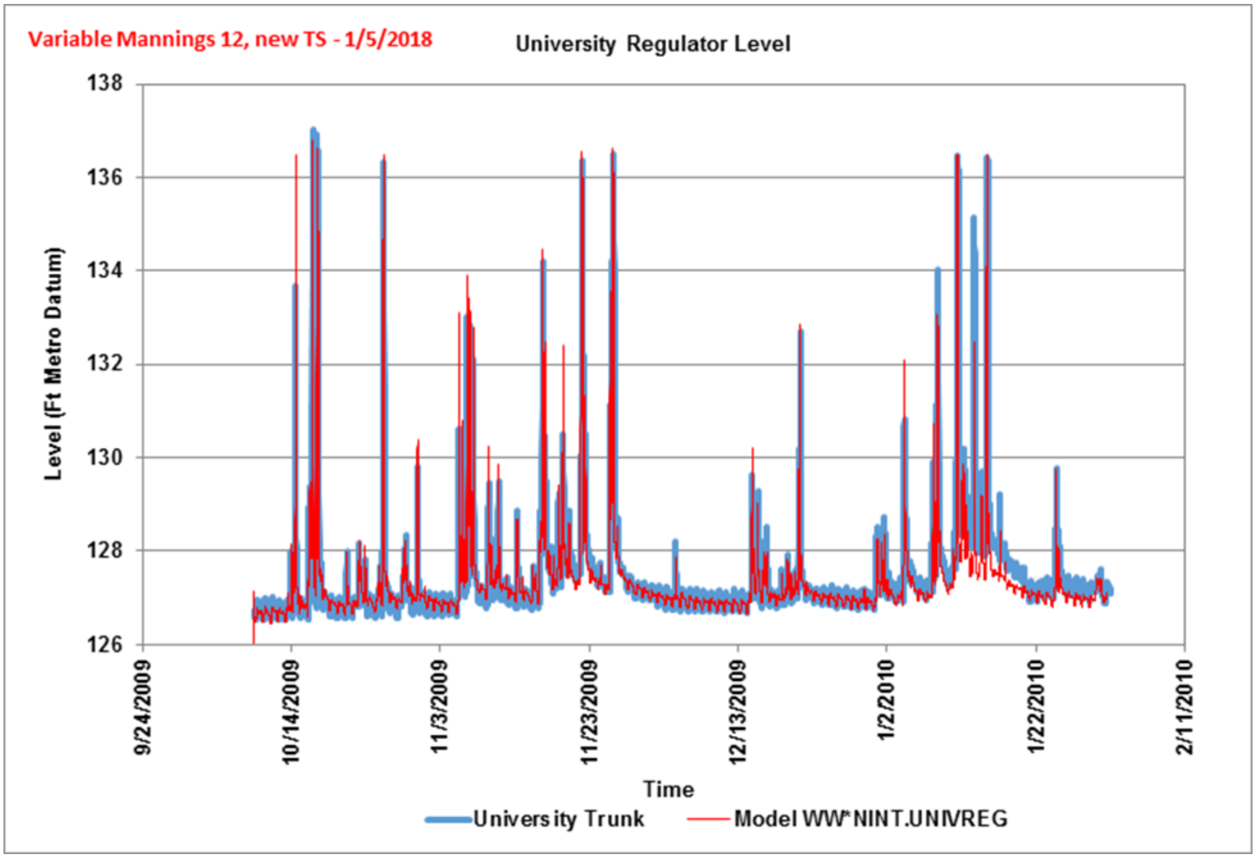


Figure 13 – Water Surface Elevation in the University Regulator Trunk

This graph shows levels upstream of the University Regulator Gate. The model is accurately predicting measured levels, indicating that all the factors, inflow, downstream level, and gate settings are working together well in the model. The model is replicating a good fit for a majority of events.

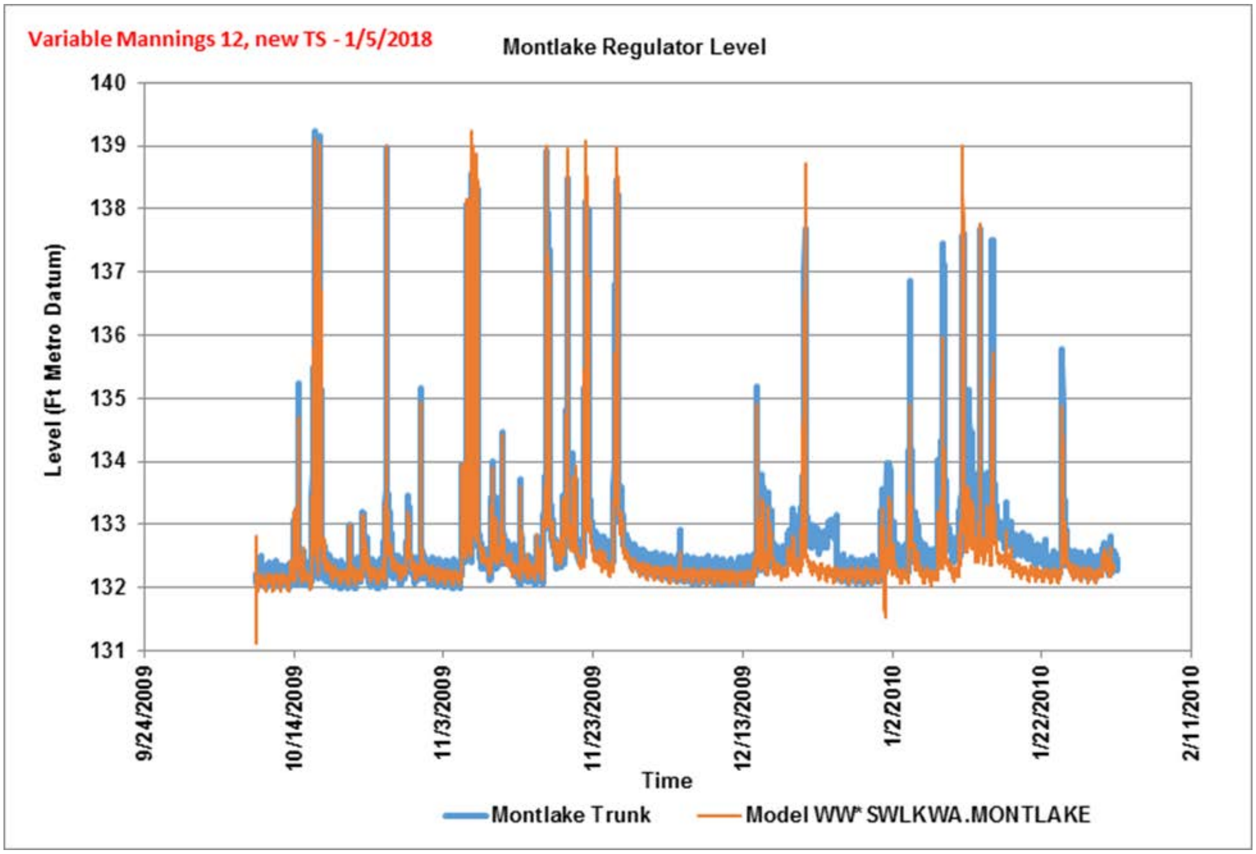


Figure 14 – Water Surface Elevation in the Montlake Regulator Trunk

Figure 14 shows the water level upstream of the Montlake Regulator gate. WTD staff identified issues with the SCADA level sensor at this location in late 2009, and repairs were made to the sensor by September 2010. The bubbler tube had a leak that was evident only at higher water levels. The measured (blue) levels did not get up to the overflow level, but the model indicates it should have, in December 2009 and January 2010 (as shown in Figure 14).

Flow Calibration Graphs

Figures 15 through Figure 20 show the comparison of simulated flow in the calibrated North Interceptor Model with the estimated flows from flowmeters or flow calculations based on water level and gate positions in the North Interceptor system.

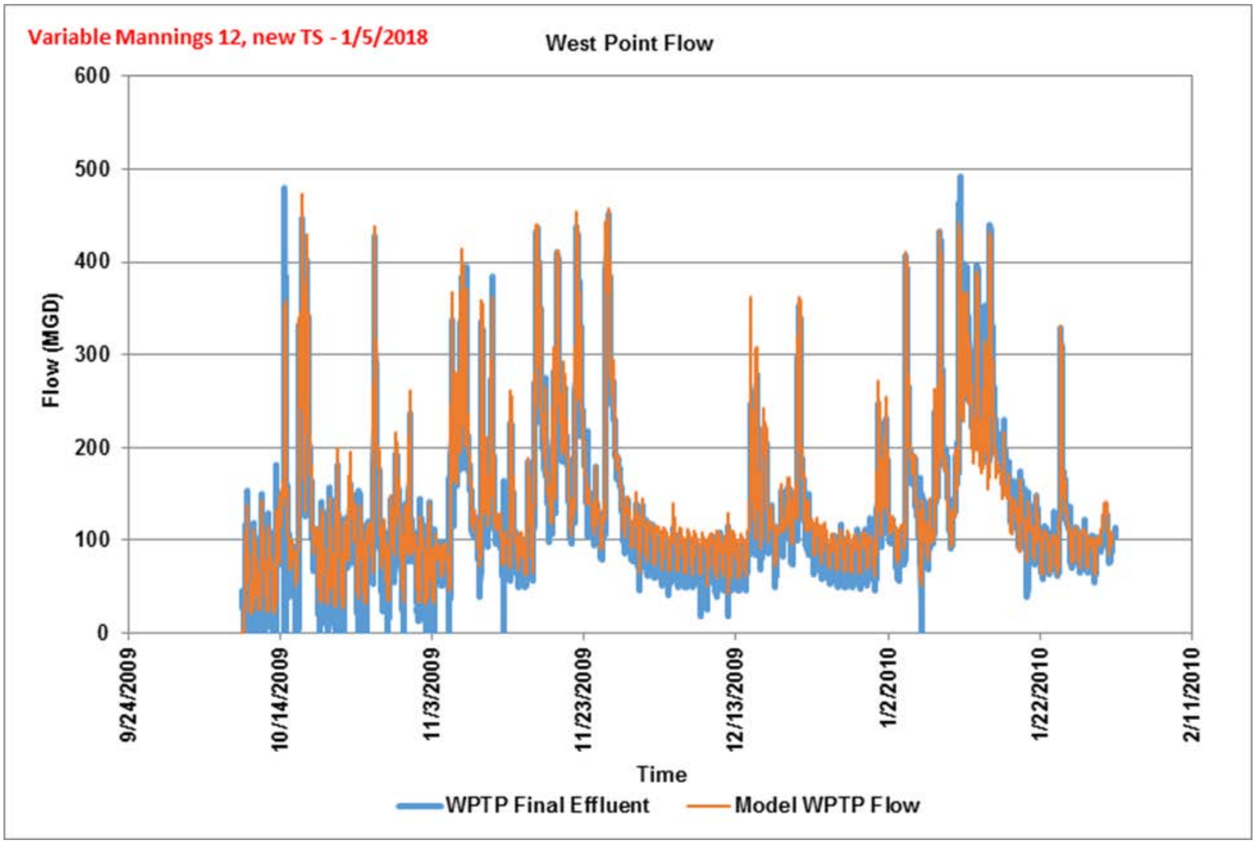


Figure 15 – West Point Treatment Plant Flow

This graph compares modeled flow to the SCADA flow calculated for West Point Final Effluent. All the possible West Point flow estimates were analyzed (sum of primary flows, sum of secondary flow and secondary bypass flow and the final effluent flow), and the final effluent flow estimate provided the most accurate results, requiring little adjustment to the model.

West Point capacity may extend above 440 MGD, but it is unlikely to approach the 500 MGD level in the graph, according to KC West Point Treatment Plant staff.

This graph shows that the inflows to the model and the hydraulic model are tuned to have a good approximation of the flow to the plant. Having the correct flow is both a necessary condition and a result of tuning the friction and minor losses in the hydraulic model.

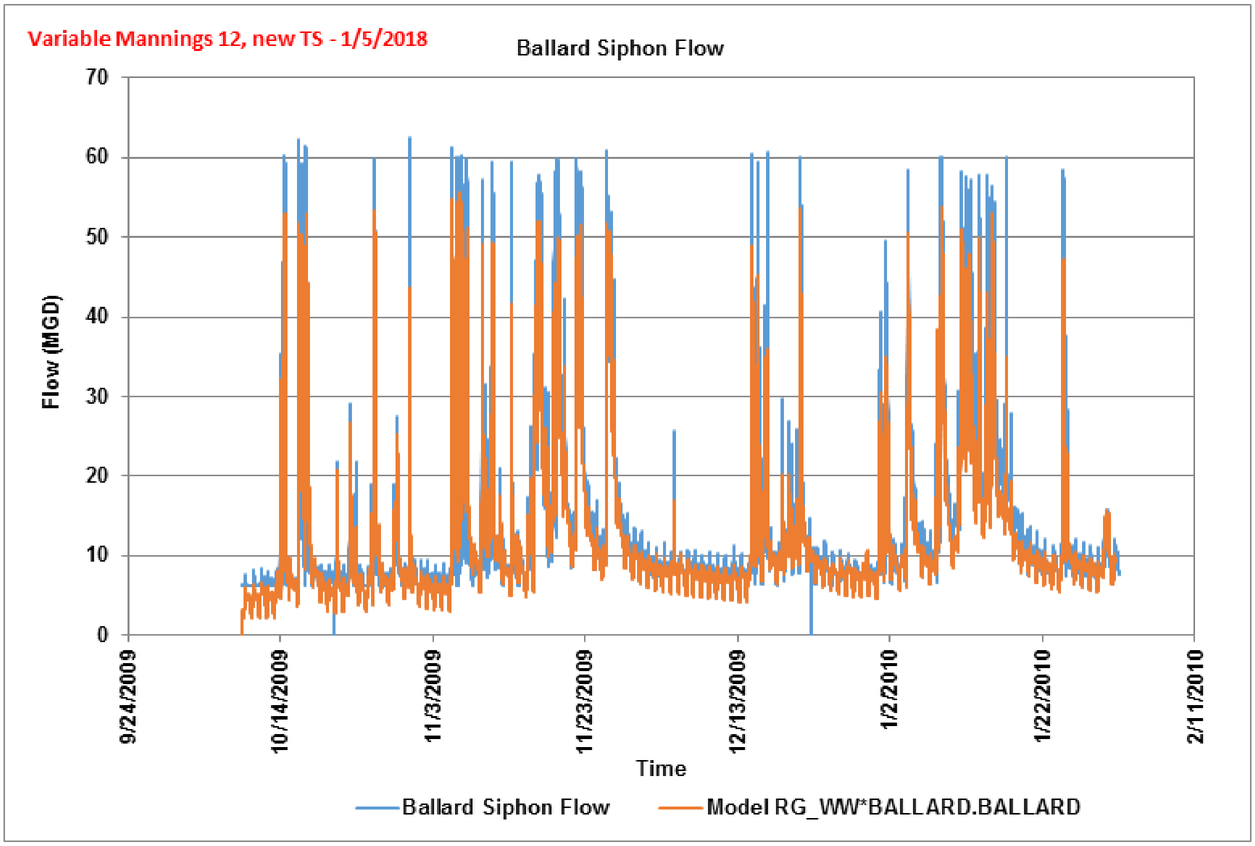


Figure 16 – Ballard Siphon Flow

Figure 16 compares flows through the Ballard Siphon. The SCADA flow calculation labeled "Ballard Siphon Flow" in the figure, is based on an upstream trunk level, a downstream interceptor level, and an orifice calculation with an assumed orifice coefficient. However, the actual siphon flow is also impacted by the friction of the siphon, so the SCADA flow calculation may be high. The upstream models sending flow to the Ballard Regulator and siphon have been calibrated to local meters, which provide the best check on flow to the siphon.

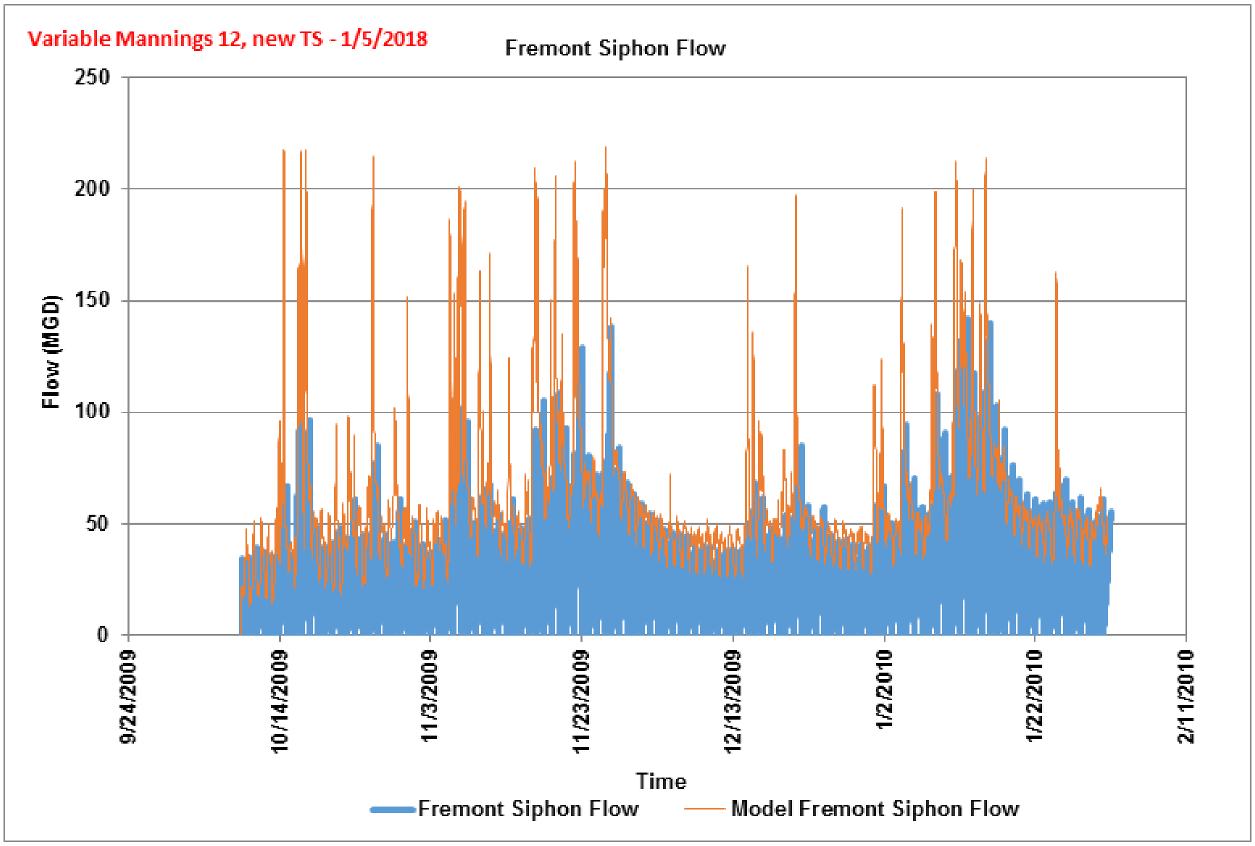


Figure 17 – Fremont Siphon Flow

The SCADA Fremont Siphon flow is a calculated flow based on the upstream level, downstream level and a flow lookup table for the siphon. The level graph downstream of the siphon reads high (higher than other upstream sensors on the Central Trunk during storm events), so the issue with the peaks is a SCADA issue, not necessarily a model issue. The calculated flow is not considered accurate, so the model was not adjusted to match the flow in this graph.

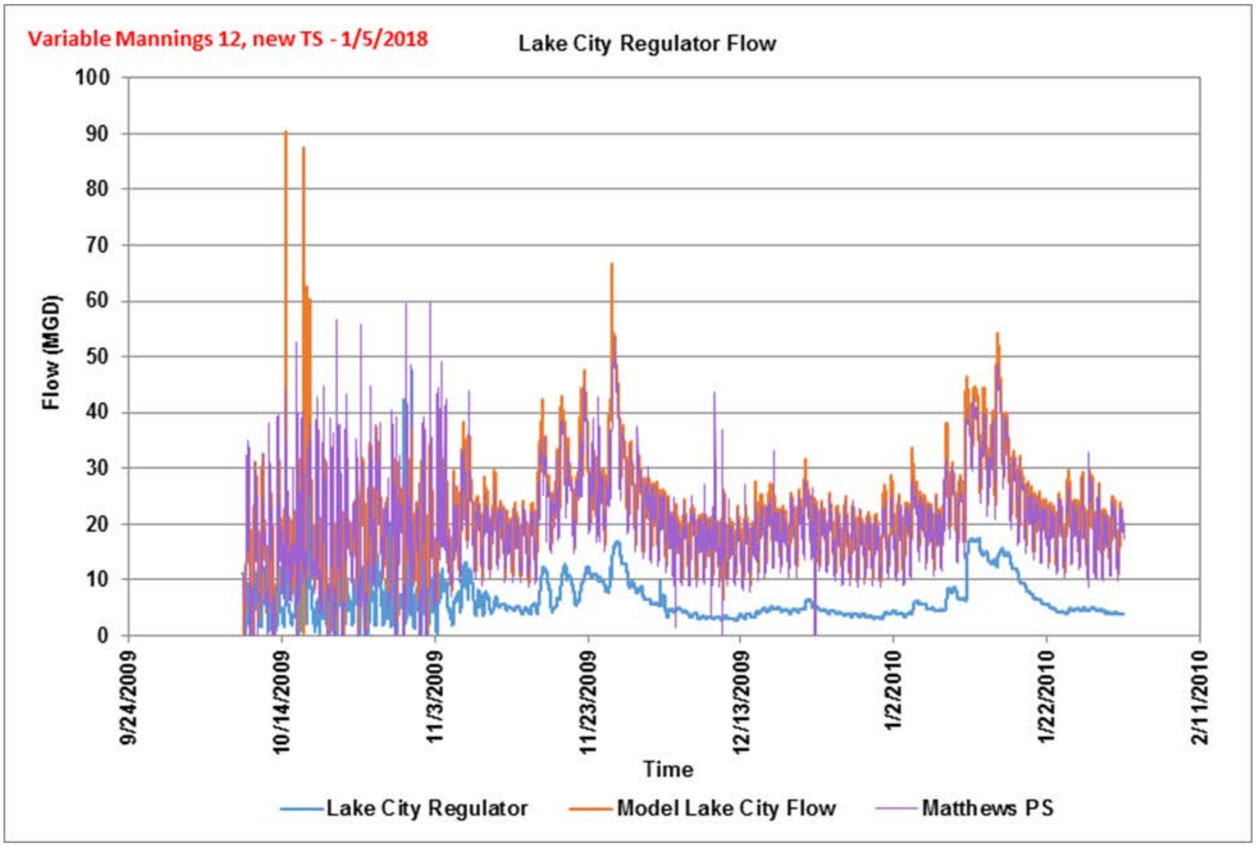


Figure 18 – Lake City Regulator Flow

The Lake City Regulator Flow is an estimate of flow through the regulator gate based on upstream and downstream water levels and on the regulator gate position. Although the level sensor used for the Lake City Regulator flow calculation has issues, the model flow can be compared to the Matthews PS SCADA of pumped flow, and it accurately matches. This is reasonable as SCADA data was used for the calibration inflow. The model spikes above the Matthews inflow are additional flow from a link to the Green Lake Trunk in the Ravenna area. When levels are high in the Green Lake Trunk some flow can shift over to the Lake City Tunnel.

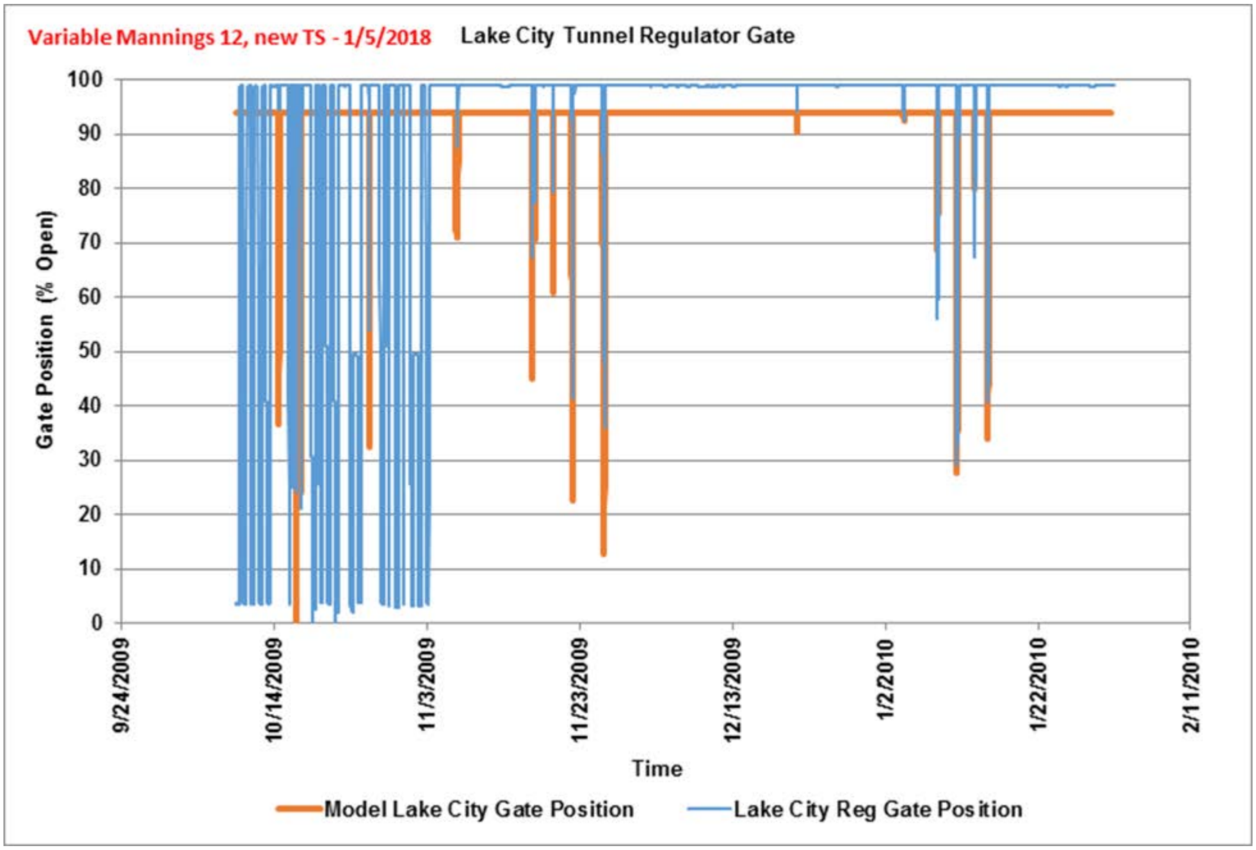


Figure 19 – Lake City Tunnel Regulator Gate Activity

Lake City Tunnel Regulator Gate is driven mostly by the level at the Canal Street Weir unless the level upstream of the gate at the regulator gets high. If you look at the level graph for Lake City Regulator, you can see that problems with that sensor are driving the excessive number of gate closures during the initial part of the calibration period. After that initial period, the model and the measured gate activity tend to correlate well. The model may drive the gate faster in some events than the measured gate movement shows. However, the model gate speed was adjusted to match the speed shown in more recent gate activity. In order to construct a model that predicts current system conditions, the calibration process allowed for slightly higher gate speed in the model.

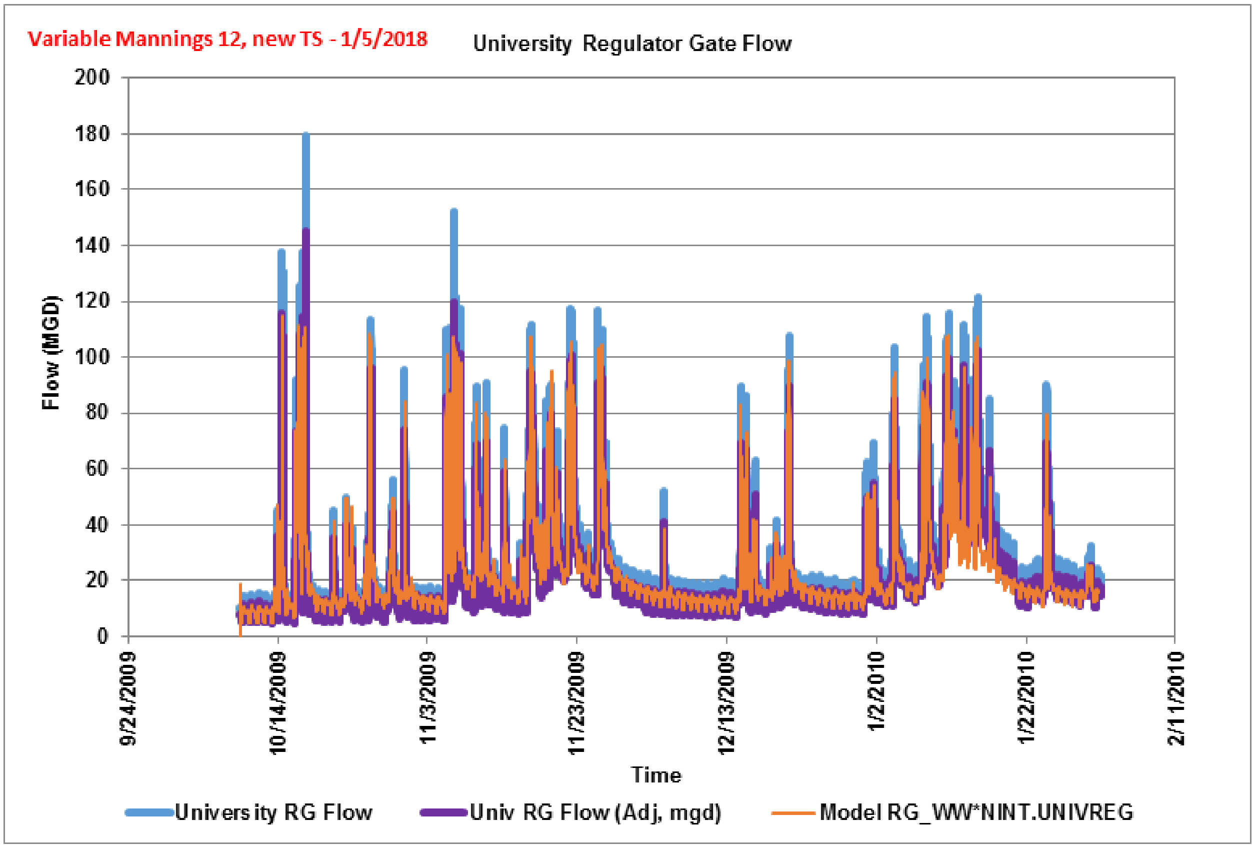


Figure 20 – University Regulator Gate Flow

The University Regulator Gate SCADA flow is a calculation using the upstream and downstream levels and the gate position as well as gate characteristics. As part of this work the gate characteristics were adjusted in the SCADA calculation to better fit new flow data obtained from a number of upstream portable flow meters. The model inflows were tuned to fit those same portable meters. This provides a much tighter calibration in this area with much less uncertainty.

CSO Calibration Graphs

Figure 21 through Figure 26 show the comparison of simulated overflow in the calibrated North Interceptor Model with flow calculations based on water level and/or gate positions in the North Interceptor system.

Overflows are the remainder of flow that the system cannot convey. Uncertainty tends to get concentrated in that remainder of flow. Overflows are also the part of the flow under investigation when evaluating CSO control projects.

It is improbable that any model constructed will show all the same CSO events and event volumes due to variability in rainfall over the basin as well as inaccuracies in measuring flows and overflows. The model is tuned to match event frequency and overall volumes even if some events are overestimated and some events are underestimated.

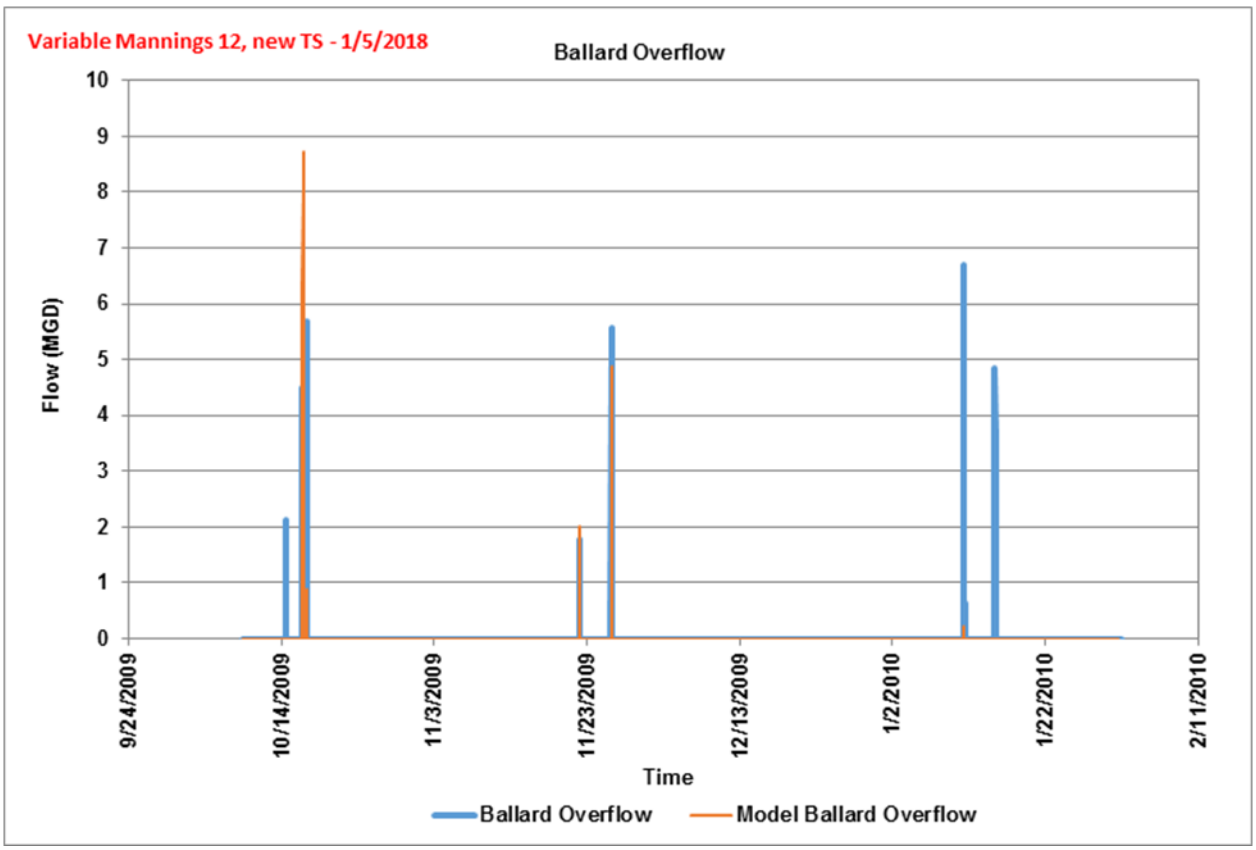


Figure 21 – Ballard Regulator Overflow

The variation in peak flow rates between the measured and model is significant; all the events are matched except for the last storm. Also note that the overflow rate is about a tenth of the conveyed flow.

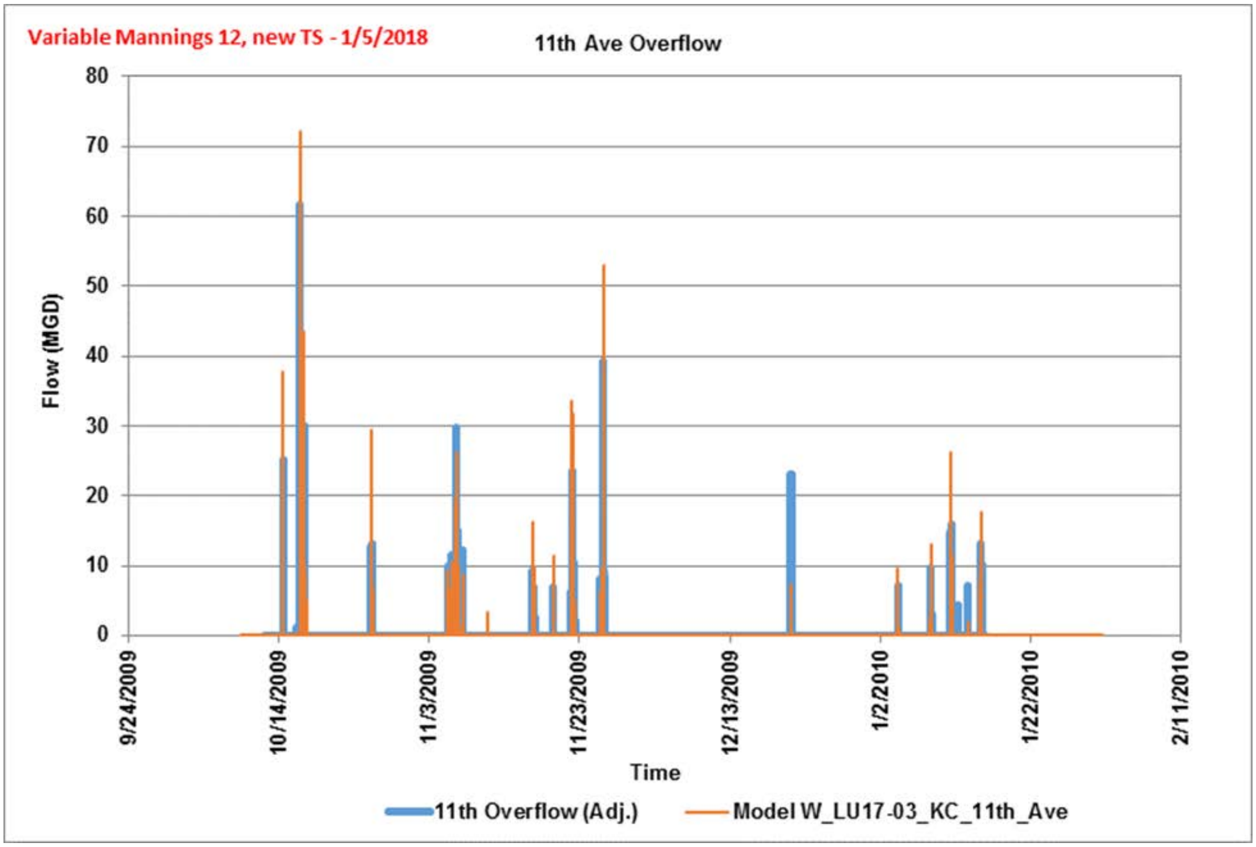


Figure 22 – 11th Ave NW Overflow

Collection of significant portable meter data at 11th Ave NW provided resolution to conflicts in information and provided data to recalculate the overflow rates based on a better knowledge of flow through that facility.

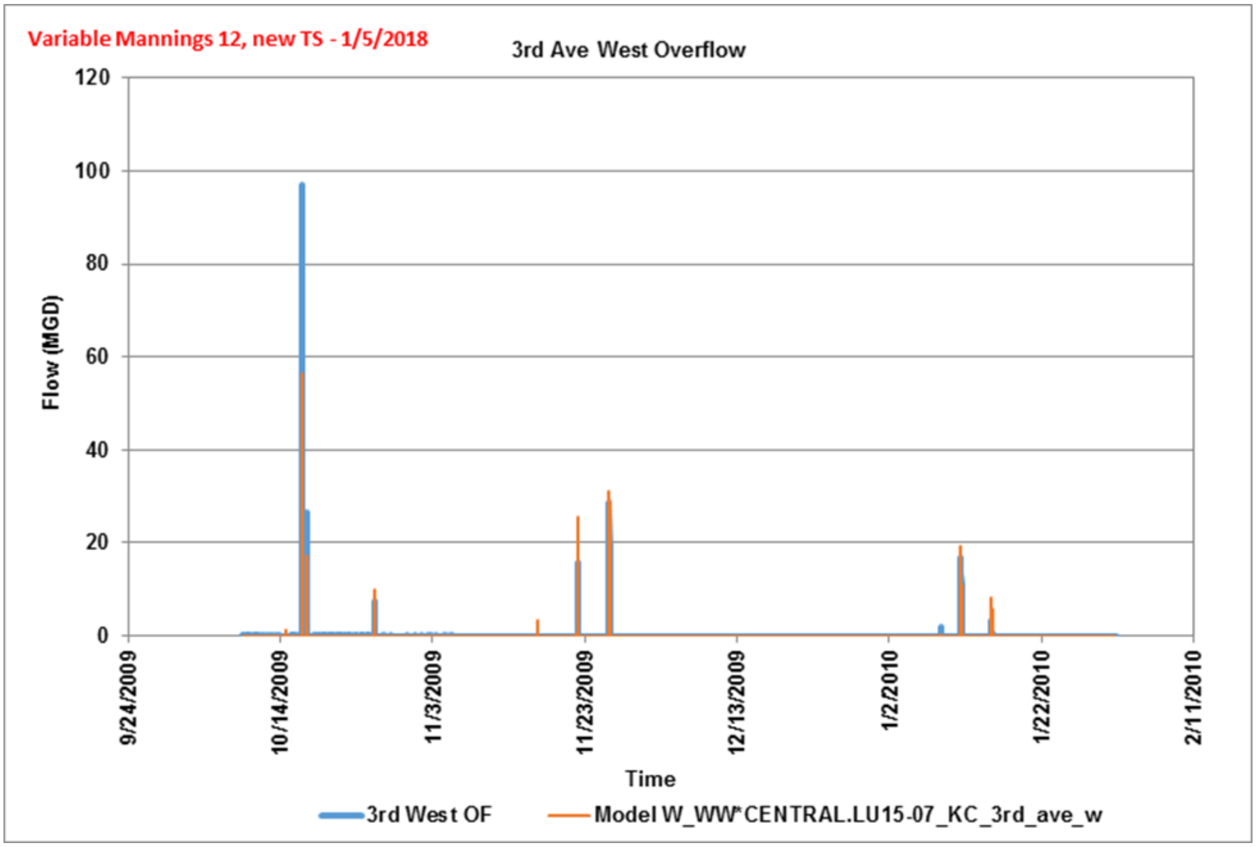


Figure 23 – 3rd Ave West Overflow

This graph shows a good match to the event count and a good match to the calculated flow rates for smaller events. Note that the calculation of measured flow does not account for the partial occlusion by rip-rap at the end of the outfall. That likely explains the significant difference between the model and the weir calculation for measured data in the largest event. Note that the model for the upstream area has been rebuilt in MIKE Urban in the past decade and was further checked and recalibrated due to a series of projects along the Central Trunk since then. The model indicates that a simple free flow weir calculation can result in high measured levels during large storm events.

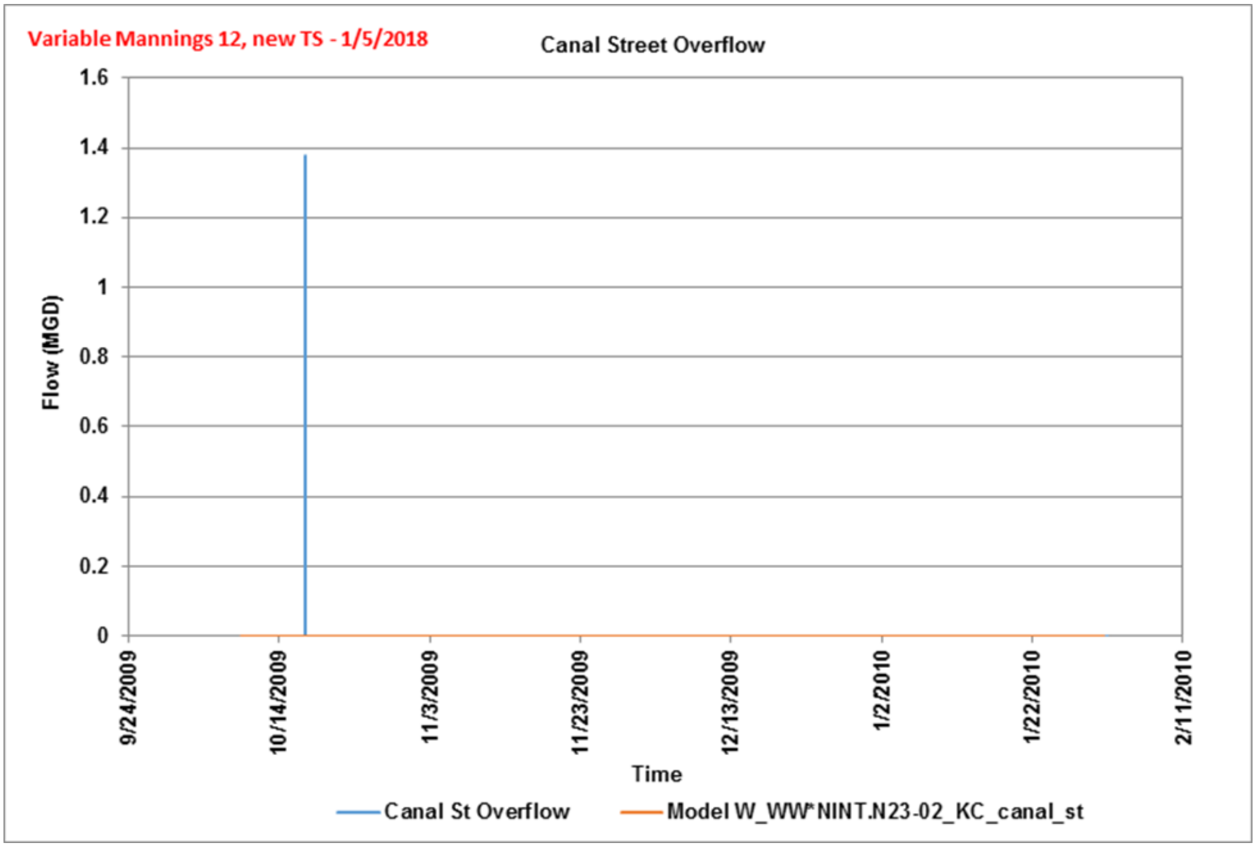


Figure 24 –Canal Street Overflow

Canal Street does not overflow often; it is controlled to less than 1 event per year, on average. Unfortunately, the SCADA level at Canal Street appeared to have quality issues during the storm event shown above. Overall, calibration was focused on trying to match the level at Canal St and allowed the level to be slightly high in the model as the one overflow could not be matched with compromised measured data quality. Also note that this possible CSO event shows a very small overflow rate compared with the high flow rate in the North Interceptor.

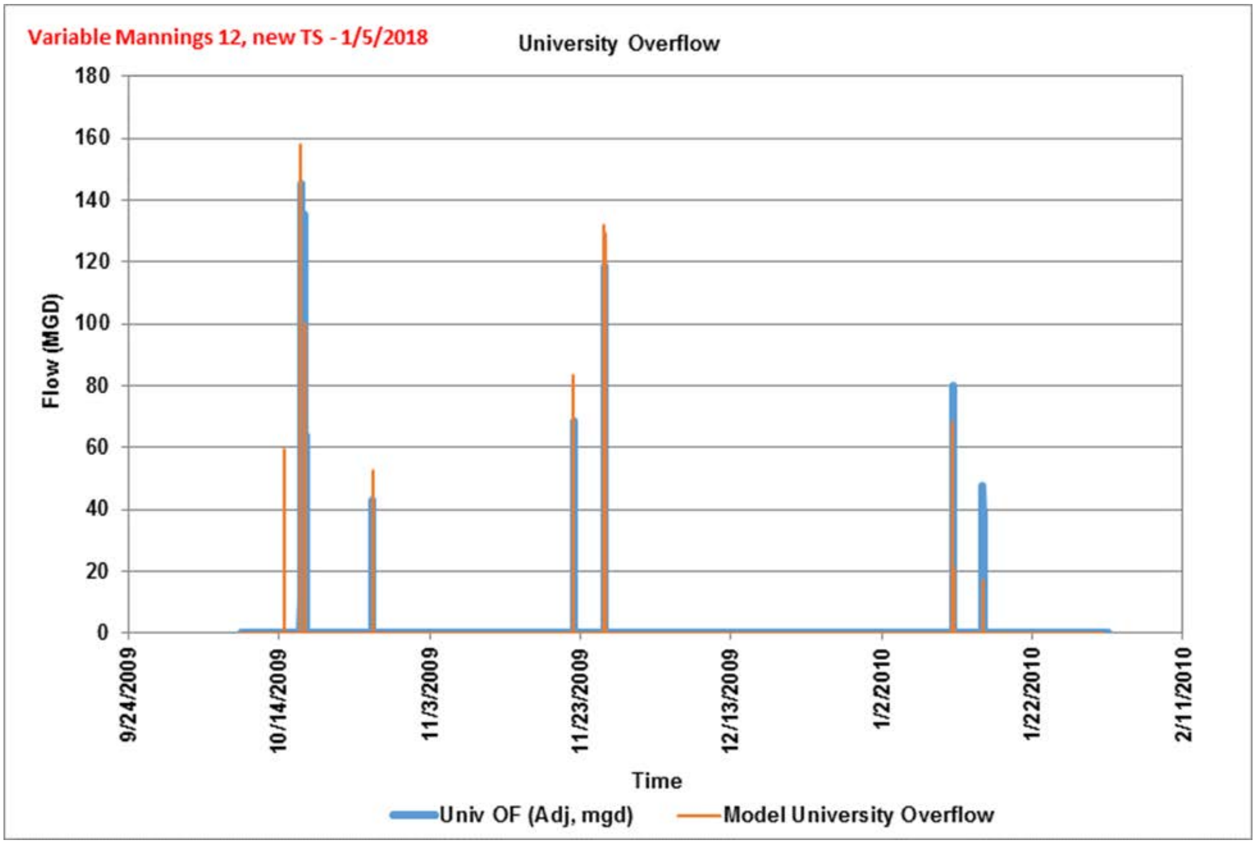


Figure 25 – University Regulator Overflow

Generally, the modeled overflow tends to match the measured overflow reasonably well at University. Note that the SCADA calculation is based on the upstream level and the outfall gate position, the gate and the weir characteristics. The outfall gate coefficient and weir coefficient were adjusted based on a better understanding of inflow provided by the upstream portable flow meters.

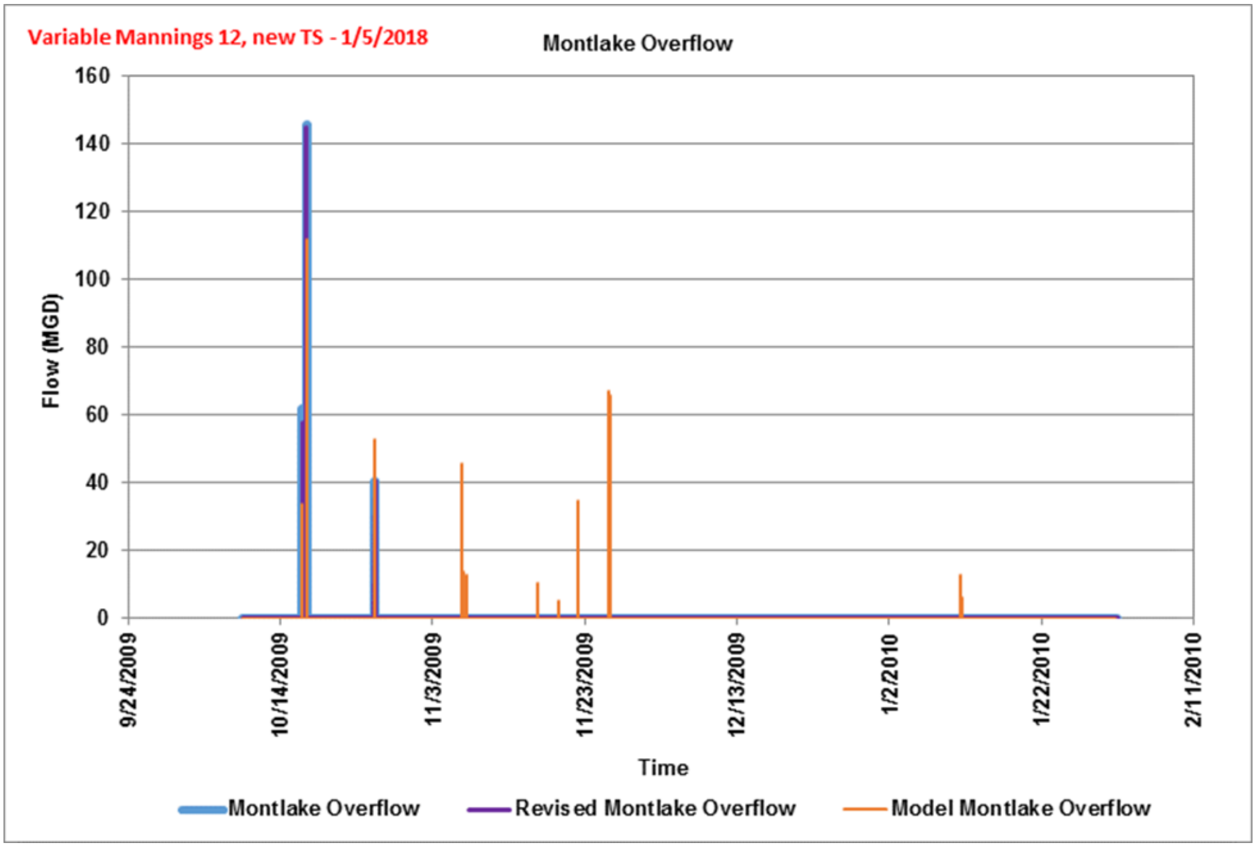


Figure 26 – Montlake Regulator Overflow

As was noted in the level graphs, the Montlake level sensor upon which measured overflow calculations were based failed to match peak levels due to a leak in the bubbler tube part way into the calibration period. So, the model shows an approximation of the missing overflows.

Note that the gate characteristics used in the measured overflow calculation may be in error. Several portable meters are going to be deployed to gain data that help to better understand the gate and weir characteristics and refine the measured overflow calculation. Based on the results at University, there may be a similar drop in the calculated overflow from measurements at Montlake. So, the above graph represents a snap shot during the ongoing system calibration work.

Conclusion

The North Interceptor MIKE Urban model that was constructed and calibrated to flow and level data obtained from SPU and KC during October 2009 – February 2010 reasonably replicates the measured and estimated flows, levels, and overflow data from that period. The model represents the hydraulic performance of the North Interceptor system, as demonstrated in the calibration graphs, and is suitable for use with the Ship Canal Tunnel Model in simulating a long-term rainfall record.